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LECTURES AND DISCUSSIONS BY PROFESSOR N. N. AMBRASEYS ON ENGINE--ETC(U)
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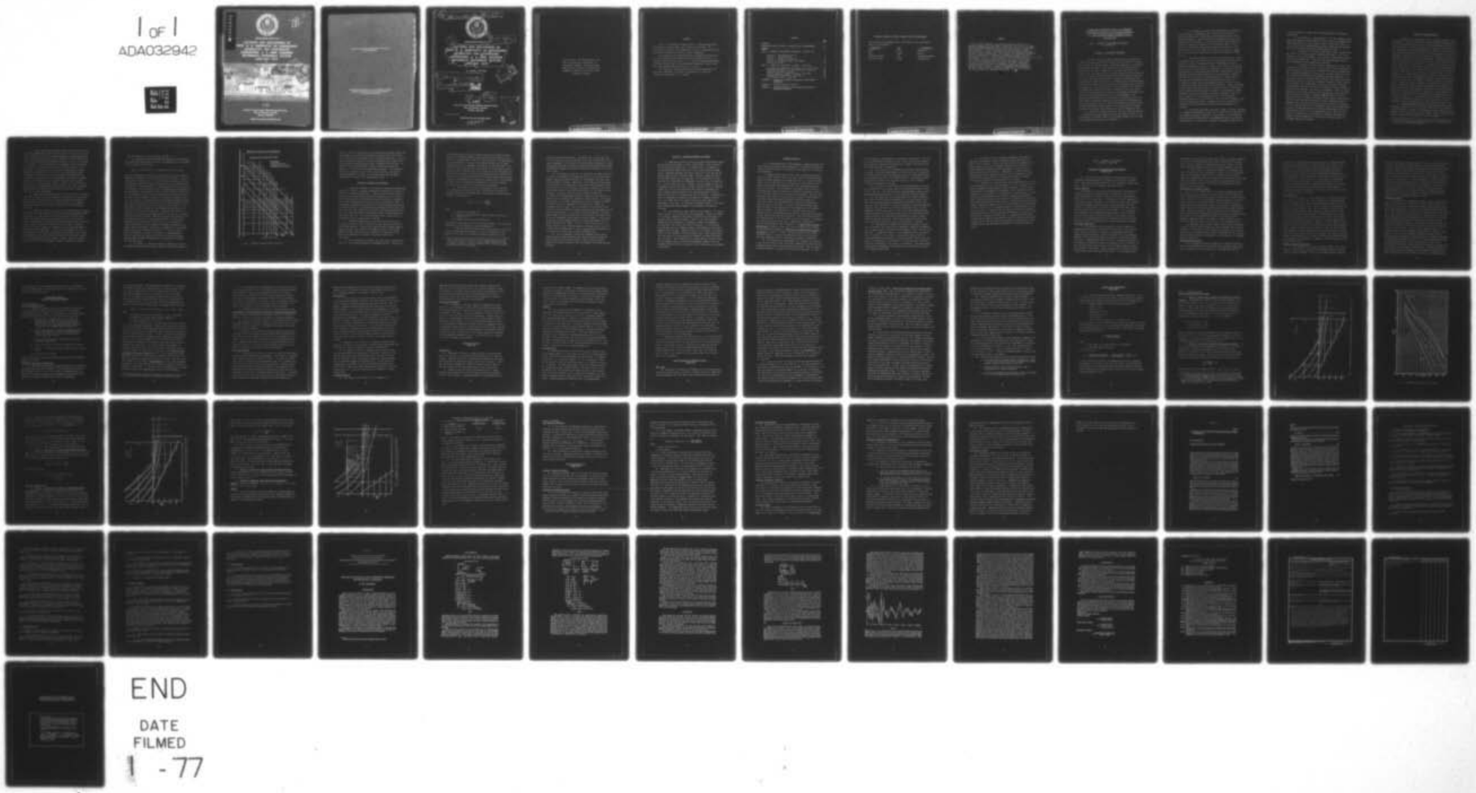

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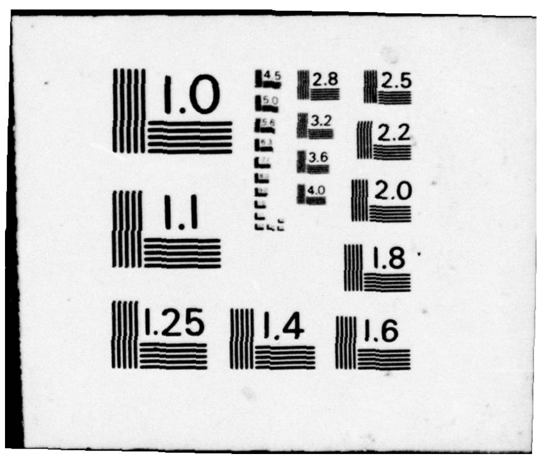
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MISCELLANEOUS PAPER S-74-15

LECTURES AND DISCUSSIONS BY PROF. N. N. AMBRASEYS ON ENGINEERING SEISMOLOGY AND EARTHQUAKE ENGINEERING, U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION APRIL-MAY 1973

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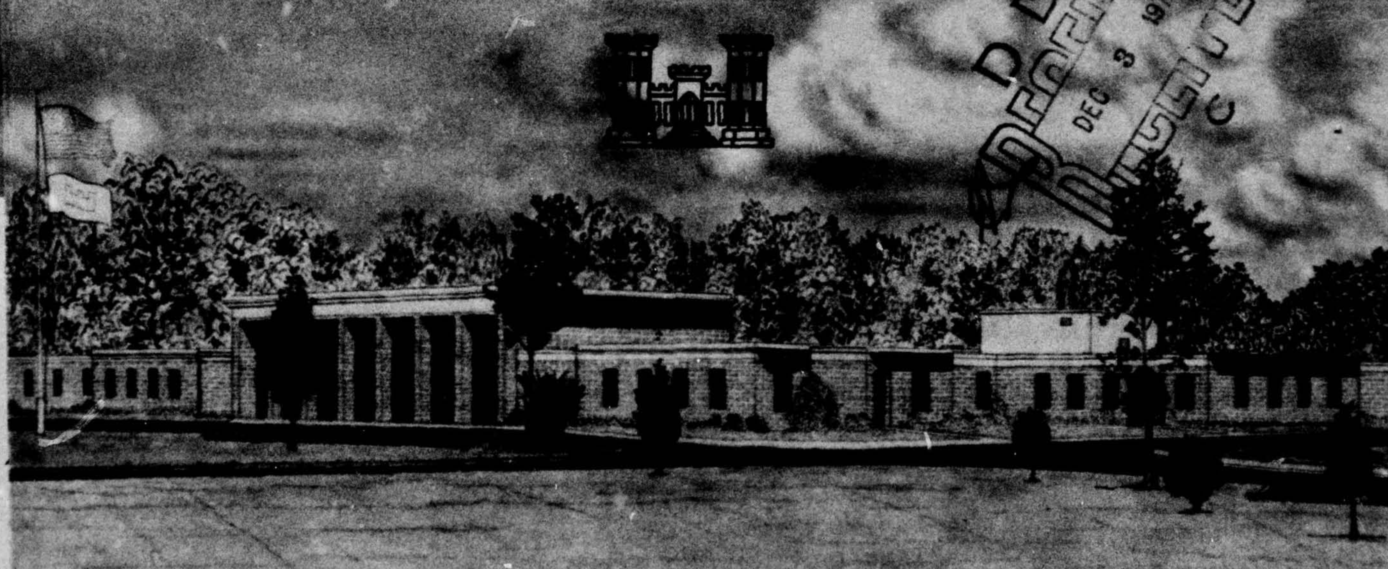

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by

S. J. Johnson, L. W. Heller

(10) Stanley J. Johnson
Lyman W. Heller

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FOREWORD

Prof. N. N. Ambraseys lectured at the U. S. Army Engineer Waterways Experiment Station (WES) from 27 April to 4 May 1973. His lectures were attended by representatives of Corps of Engineers District and Division offices and the Office, Chief of Engineers.

His lectures and the discussions that followed are included herein. This compilation was prepared under the supervision of Mr. S. J. Johnson, Special Assistant, and Dr. L. W. Heller, Earthquake Engineering and Vibrations Division, Soils and Pavements Laboratory. Chief of the Soils and Pavements Laboratory was Mr. J. P. Sale.

The Directors of WES during the lectures and the publication of this report were BG E. D. Peixotto, CE, and COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
inches per second	2.54	centimeters/second
feet per second	0.3048	meters/second

SUMMARY

This report summarizes lectures and discussions by Prof. N. N. Ambraseys, Imperial College, London, during his visit to the ~~U. S. Army Engineer Waterways Experiment~~ this Station (WES) from 27 April to 4 May 1973. Prof. Ambraseys' lectures on engineering seismology were attended by representatives of Corps of Engineers District and Division offices and the Office, Chief of Engineers. These lectures are summarized ~~in Part I herein~~. Discussions of geological and geophysical aspects and field investigations relevant to earthquake were held with WES personnel. Topics included The sequence of the discussions followed a list of discussion items that had been previously submitted to Prof. Ambraseys. Discussions also covered the application of laboratory test data in analytical procedures, current research in earthquake engineering seismology at Imperial College, Ambraseys' method for computing dam displacements, and other topics related to earthquake engineering. The discussions with WES personnel are summarized in Part II.

LECTURES AND DISCUSSIONS BY PROF. N. N. AMBRASEYS

ON ENGINEERING SEISMOLOGY AND EARTHQUAKE ENGINEERING

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION

APRIL-MAY 1973

PART I: LECTURES ON ENGINEERING SEISMOLOGY
30 APRIL 1973

Lecture 1: Earthquake Generation

1. The first topic, earthquake generation, is essentially one in which engineering seismology is brought into the picture to focus on fault behavior during earthquakes. Ambraseys showed slides of relationships between the number of earthquakes that had occurred along different faults and time. Perhaps one of the best data sources for recorded history of earthquakes is in areas where old cultures kept good records. Some areas have an earthquake history that extends back some 3000 years; good maps, pinpointing the location of earthquakes, were developed by the tenth century A.D. A comparison of two different areas was made which showed that the occurrence of earthquakes may be periodic in one area, but not in another area. Thus, the number of destructive earthquakes per century may be fairly consistent in one area, but in other areas there may be short periods of considerable earthquake activity and long periods during which only a few earthquakes occur. Ambraseys concluded that even 20 centuries of recorded history is not long enough to establish recurrence relationships for earthquakes and that, in his opinion, one must consider the geological setting and the project importance for which seismic risk is being determined before any kind of recurrence relationship can be used for design purposes.

2. Some of the surface evidence of historic earthquake activity is in roads built by the Romans which have been offset as much as 6 m horizontally. Buildings erected in the fourth century were cut in two by fault movements and had to be abandoned.

3. It is generally conceded that earthquakes are generated by slip along faults. Frequently, most of the slip occurs along old established fault zones. Only rarely are new faults involved in generating earthquakes. Large destructive earthquakes are the result of movements along long faults. A long fault releases more energy than a short fault because the duration of the shaking is longer. Thus, a short fault break may produce peak acceleration and velocity values that are equivalent to a longer fault break, but, because the shaking due to the short fault break does not persist, the cumulative damage is less.

4. Ambraseys reviewed the strike-slip fault model and related this model to the fracture that occurs in a laboratory strength test of concrete specimens. Ambraseys noted that faults may move without generating earthquakes; he cited a fault in India that has been moving 10 cm per year but has produced no earthquakes. He has often observed weak materials on steep slopes that have been faulted upward; thus the movements during the faulting must have been very gentle. Moreover, there has been no evidence of vitreous material on exposed faults; high velocity faulting would have produced melting of the rock.
On large-scale maps, faults are sometimes shown as continuous lines; on small-scale maps, these faults are discontinuous. Aerial photographs usually show discontinuous patterns of ground cracking. Faulting can also be found from descriptions of changes in old groundwater systems. Groundwater systems are important earthquake indicators. Tree roots may be pushed up out of the ground in zones of fault movements. Ambraseys feels that 60 to 90 percent of the total fault movement is due to slow creeping of the fault. Sometimes creep can be measured in a fault zone with strain meters installed after the earthquake has occurred.

5. Ambraseys said that it was time to begin to design structures to accommodate displacements rather than to resist stress, especially for the seismic stability of structures. He said that it is necessary to develop factors of safety based on displacements rather than stresses and that there is a real need to identify the important signals in the

noise of research literature that is being developed in earthquake engineering.

6. In response to a question by Mr. N. A. Dixon, Ambraseys suggested that dams be designed with wide filter zones to accommodate movements that would occur in the body of a dam. He said that there is a need to work with the dimensions and problems caused by crack patterns that would be formed during displacements within the dam.

7. In response to a question regarding the short recorded seismic history in the United States, Ambraseys said that it just is not possible to force a seismologist to put numerical ground motion answers on historical damage data. Collecting and reviewing damage data for past earthquakes is time consuming and expensive.

8. In response to another question, Ambraseys said that if the foundation of a structure is very good and there is little likelihood of static instability then there will probably not be much earthquake damage. Ambraseys suggested that the site for a project should not be selected before the foundation considerations have been evaluated. For the seismic design of the Aswan Dam, 15 percent g horizontal forces were applied to the crest of the dam; a parabolic variation of this horizontal acceleration was assumed from the crest to the base of this dam. Filter material has been stockpiled at the dam site and is available to plug any cracks that may occur during an earthquake. There is a report available on the seismic design of this dam published by the Water and Power Association of Egypt. Ambraseys suggested that the Corps of Engineers should be studying self-healing filters rather than the application of finite element procedures for seismic stability analysis. Ambraseys noted that there have been several dam failures in Japan and a few of these have been due to earthquakes; there is not enough empirical data to answer questions such as how much displacement within an earth dam will cause it to fail progressively due to erosion or overtopping. Ambraseys said that the State of California and Japan are probably the most progressive in attacking the research aspects of earthquake engineering problems.

Lecture 2: Ground Response

9. Ambraseys next addressed the topic of ground response during earthquakes. He first observed that intensity maps are essentially useless for design purposes. As early as 1952, Hershberger warned that intensities were not related to ground accelerations in any logical manner. Ambraseys' studies have indicated that there is no reliable correlation between peak ground acceleration and the magnitude of a causative earthquake. Formally, there is no upper bound for ground acceleration; particle velocity, however, has an upper bound. Depending on the material in the foundation, acceleration values are limited by the strength of these materials and their ability to transmit acceleration from base rock to the surface of the ground. The material in the foundation limits the energy that can be transmitted in the same way that the weakest link in a chain limits the force that the chain can endure. Thus, a magnitude 3-1/2 earthquake may generate a peak particle velocity of 3 to 4 fps* and a magnitude 7 earthquake may produce the same peak velocity in the near field area. The duration, however, depends on the magnitude. In addition to this transmission consideration, Ambraseys said that the energy arriving at the structure depends on the radiation of the wave motion into the foundation material and into the dam itself; this is the response problem that is important for the analysis of the safety of an earth dam. Due to confinement, the maximum particle velocity at a depth of 3 to 4 km might be about 500 cm/sec on the basis of indirect computations; near the surface, because the confining pressure is less, the maximum particle velocity would be only 4 fps.

10. Ambraseys warned that data from nuclear and high explosive experiments should not be used to measure upper bounds for particle velocities because a nuclear or high explosive source generates energy, whereas a fault releases stored strain energy. Thus, an explosion is a source of energy and an earthquake is a sink of energy.

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* A table of factors for converting British units of measurement to metric units is presented on page ix.

11. Many building codes prescribe that structures on soft ground should be designed with a higher seismic coefficient than structures on rock. Ambraseys feels this is wrong; buildings on soft ground are damaged by earthquakes because the initial stability of these buildings is low and the earthquake easily damages structures that are on the threshold of failure. He feels that much of the damage is due to settlement of structures on soft ground and not due to vibration caused by the earthquake. He pointed to the result of the earthquake in Niigata, Japan, as an illustration of the loss of soil support which caused structures to settle and tilt. These buildings were shaken most severely during the early part of the earthquake and suffered no damage, but the loss of support in the foundation resulted in tilting that rendered many of the buildings unusable. While a soft layer may fail during shaking, the development of pore pressures and their slow migration may cause a foundation failure minutes or even tens of minutes after the end of the earthquake.

12. Where ground cracking has been found in the field, Ambraseys noted that the peak horizontal acceleration coefficient for these sites would be equal to the ratio of the unconfined compressive strength to the effective vertical stress in the zone of failure. For normally consolidated materials, the seismic coefficient at failure would be about 0.3.

13. Ambraseys has noted during his field investigations that many weak structures in the vicinity of a fault scarp are not damaged during earthquakes. In other cases, he has observed stones that have been thrown out of their sockets and were resting on top of the ground; he feels that ground motion amplification has caused these stones to be dislodged. Heavy structures were affected less than light structures. Also, from his field experience, Ambraseys noted considerable damage to reinforced concrete buildings. He feels that much damage was due to poor construction rather than poor design and inadequate seismic stability judgements. Also, many reinforced concrete buildings were cracked due to settlement before the earthquake occurred. Analysis of damage to brick walls after the earthquake at Skopje, Yugoslavia, indicated

that accelerations of 0.8 g must have occurred.

14. Ambraseys restated his empirical equation for the relationship between the peak particle velocity, the magnitude of an earthquake, and the distance from the focus

$$\log_{10} v = 4.02 + 0.72M - 0.5 \log_{10} (11.5M - 53.0) - \log R$$

which was developed for epicentral distances of 10 to 150 km, Richter magnitudes from about 5 to 7, and velocities to 140 cm/sec. A plot of the equation is shown in fig. 1. For sites that are located less than 10 km from the epicenter, the peak velocity will not be influenced by the magnitude of the earthquake. For sites located more than 10 km from the epicenter, the peak velocity will be influenced by the magnitude of the earthquake. His discussion of this equation followed (see Appendix A). He emphasized that use of this equation should include a check to determine if the subsurface materials can transmit the computed velocity. With respect to frequency content of earthquakes, Ambraseys noted from his field experience that at one end of a fault chimneys on buildings would be thrown down, but at the other end they would not. He believes that this is because of the difference in the frequency of ground motion at each end of the fault. Ambraseys feels that it is difficult to prescribe an earthquake because of the variety of spectral contents that can be involved. Ambraseys explained the derivation of his empirical equation for relating peak particle velocity, earthquake magnitude, and distance from the focus. He said that the equation was similar to those suggested by Newmark, Rosenbluth, and Esteva; however, Ambraseys' values will be perhaps two or three times greater than those recommended previously. To obtain a value for ground displacement, one must first normalize a recorded ground motion so that the peak velocity will match the design earthquake. Then determine the peak displacement by integrating the normalized velocity time record.

15. In answer to a variety of questions, Ambraseys said that settlements and twisting caused most reinforced concrete structures to

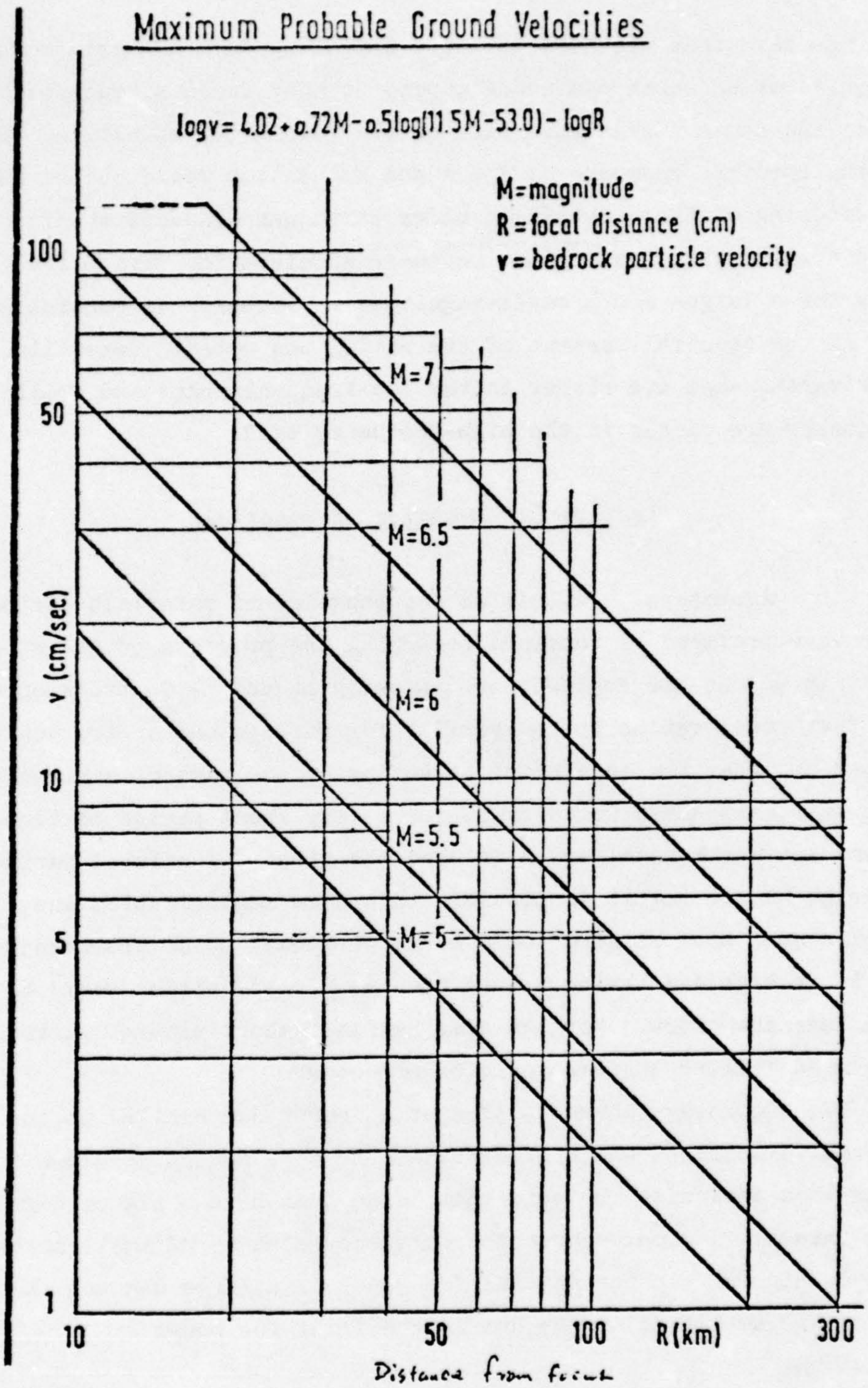

Fig. 1. Maximum probable ground velocities

fail. He said that the need to use a traveling wave analysis method for the design of an earth dam would depend on many factors, such as the size of the dam and the properties of the foundation materials. He said that the spectral response of a dam and foundation would change due to the softening of these materials under earthquake-induced strains. He showed a slide illustrating the response acceleration versus frequency for a large- and a small-magnitude earthquake. A considerable shift in the spectral content of the motion was noted. Generally, larger earthquakes are richer in the low-frequency band and small earthquakes are richer in the high-frequency band.

Lecture 3: Behavior of Materials

16. Ambraseys' comments on the behavior of materials during earthquakes were prefaced by comments regarding the progress of the state of the art in earthquake engineering. Looking at the El Centro record, he noted that acceleration spikes persist for perhaps one or two tenths of a second and that the accelerations acting on the base of a structure due to this earthquake would exist for a very short period of time. If the dominant earthquake period matched the first few natural periods of the design structure, it is possible that some amplification and, in the case of a dam, some sliding would occur with each pulse absorbed by the dam. For a high dam having a long natural period, slides would be expected near the crest. For low dams having a short natural period, a deep-seated failure surface would be expected.

17. Ambraseys showed slides of an earth dam excited in the horizontal direction, which illustrated the mode shapes obtained from a shear beam analysis. He said that large dams have a higher degree of safety than small dams because the frequency content of most earthquakes is higher than the natural frequency of a large dam and thus would not be amplified in the dam in the first few modes of oscillation.

18. Prof. Ambraseys illustrated the stress path to failure envelope for a soil material subjected to earthquake effects. The path to

failure depends upon the accelerations acting in the body of the dam. One way of estimating permanent displacements that can develop in the dam is by using Ambraseys' charts.* These charts have been developed for various parameters: the slope of the failure surface, pore pressure parameters A and B for embankment materials, the effective angle of internal friction of the material, the duration of an acceleration spike, the peak particle velocity that occurs during the entire earthquake, and the critical acceleration at which the sliding mass has a safety factor of one. The value of the effective friction angle and the pore pressure parameters are determined from laboratory undrained tests. To determine the peak value of the permanent displacement in the dam, different durations of the acceleration spike should be considered.

19. With respect to residual displacements in an earth dam, Prof. Ambraseys has developed an upper bound empirical equation for evaluating residual displacements based on many cases calculated with a computer. The equation is

$$\log(u) = 2.3 - 3.3\left(\frac{k_c}{k_m}\right)$$

where

u = residual displacement (cm)

k_c = critical acceleration (g) needed to reduce the factor of safety to one

k_m = maximum input ground acceleration (g)

This equation eliminates the use of the detailed charts. The source and limitations of this equation are given in Appendix A.

20. Ambraseys briefly reviewed some of his thoughts on the liquefaction of sands during earthquakes. He has done analyses of the behavior of two layers of sand having different permeabilities and

* Most of these charts were given to WES during Ambraseys earlier visits and are on file in the Soils and Pavement Laboratory. WES is trying to arrange to obtain a complete report on this procedure together with the missing graphs.

different liquefaction potential. He showed that it takes some time for the excess pore pressure developed in the deep liquefied portion of sand to propagate to the surface of the ground and be dissipated. These calculations show that the bearing capacity failures that occurred after the Niigata earthquake were caused by excess pore pressures in the foundation sands.

21. In response to questions on laboratory test methods that can be used for determining the strength properties of soils during earthquakes, Ambraseys described certain test developments that have taken place at Imperial College. Essentially, their efforts have been aimed at reproducing, in the laboratory, the acceleration environment experienced by the in situ soil during an earthquake. Ambraseys stated that their efforts have not been entirely successful, but he feels that more thinking is necessary to better define the type of tests that are needed for the earthquake situation and that the Corps should not blindly follow laboratory test procedures used by others. In one popular procedure, for example, Ambraseys noted that the number of significant stress cycles is independent of the direction in which the earthquake record is read. One could reverse the acceleration time-history for any earthquake and obtain essentially the same number of significant stress cycles. It is possible that the sequence of stresses might change the material behavior and that different conclusions would be obtained depending on the sequence of stress pulses. He feels that monotonically loaded samples with pore pressure measurements are the proper laboratory test today. If the A factor is from -0.2 to -0.3, further testing is not required; whereas a strongly positive value of A means that a strength loss on cyclic loading should be expected. He does not like the usual type of cyclic laboratory test.

22. Also in response to questions, Ambraseys stated that he thought seismic design procedures were ready for practical application, but he also said that he would not want to make a seismic analysis before good laboratory tests and adequate field data had been obtained.

Lecture 4: Planning, Hazards, and Damage

23. Ambraseys explained his concepts regarding earthquake hazards and the planning necessary to reduce these hazards. Some of the questions that must be answered are: (1) quantification of damage and (2) specifying the hazards that arise due to these quantified damage values. Ambraseys explained that public officials in the past had the same problem as public officials today. Written in Roman records are certain restrictions that buildings will not be constructed in areas that have been devastated by earthquakes; in other areas, buildings are not to be more than 60 ft tall. However, with the passage of time, encroachments on these directives occurred. Historic records in Antioch showed that this city was devastated many times by earthquakes; but people forget quickly and after two or three generations an entire practical education in earthquake risk must be repeated. As recently as 1903, an earthquake in southern Italy resulted in complete destruction. But today, most of the people there have never heard of an earthquake in this area. To understand earthquake hazards, it is necessary to observe the social impact of an earthquake on developing countries.

24. For long-term economic impacts to be minimized, it is necessary to plan for the consequences of an earthquake. One of the results of the Skopje, Yugoslavia, earthquake in 1963 was the development of improved seismic design procedures. However, some rather bizarre designs have also developed; for example, a house or an apartment building built like a ball and sited on hillsides. It is theorized that the ball will roll slightly during an earthquake and thus prevent damage to the structure. However, should it roll downhill, it could be lethal for the occupants of the ball-shaped house. Even though design codes are used, it is not practical to prescribe how these structures are to be built. Poor construction can negate skillful seismic design. In a developing country, losses from earthquakes, famine, fire, and economic problems can be as serious as loss of lives due to the collapse of buildings.

General Discussion

25. Following Ambraseys' formal remarks, he addressed himself to several questions posed by the participants. One question involved the steps that are necessary in the analysis of the stability of a dam during earthquakes.

26. Ambraseys said that the first step in attacking such problems is to make a seismic study of the area. This involves gathering appropriate data regarding (a) seismic history; i.e., a list of seismic events, (b) the locations of all epicenters, (c) the locations of all faults, and (d) data concerning the most recent movements along the faults, if possible. The reliability of epicenter locations must be determined and it may be necessary to recompute the epicenter location to be consistent with the fault causing the earthquake. When this has been done, the location of the project with respect to active faults is considered using the geometrical statistics method proposed by Prof. Allin Cornell of the Massachusetts Institute of Technology for previous events on the fault. Using the relationship between peak particle velocity, the magnitude of the earthquake, and the distance from the project to the epicenter, in conjunction with Cornell's method, a plot of the maximum particle velocity versus the return period of this particle velocity can be prepared. This relationship reflects the characteristics of the faults and the site with respect to expected peak particle velocity at the site. (Additional information on Cornell's method may be found in Chapter 27 of Dynamic Waves in Civil Engineering, John Wiley & Sons, 1971, and in a paper entitled "Engineering Seismic Risk Analysis," Bulletin of the Seismological Society of America, Vol 58, No. 5, 1968.) The return period of the design earthquake should not involve extrapolating time to more than three times the available period of observation, according to Ambraseys.

27. The second step of the analysis is to analyze the stability of the dam for strictly static conditions, ignoring the influence of the earthquake ground motion, using the method of slices. Next, the dam is statically analyzed to determine the critical acceleration k_c that

must be applied to various depths of potential sliding masses in the dam to reduce the factor of safety to unity. Thus, a relationship between the critical acceleration and the position or depth of the failure surface below the crest is established.

28. The third step is to establish acceleration time histories for design. An actual accelerogram record from the site, if available, is used and scaled to give the same peak velocity as determined from the velocity-period curve (i.e. the design velocity). The maximum or peak acceleration associated with this time history is used to compute residual displacements in the dam.

29. Step four is a calculation of the displacements of the critical sliding masses in the dam, using the previously determined values for k_c and k_m. This can be done by using Ambraseys' charts, by his empirical upper-bound equation, or by a lumped mass dynamic analysis.

30. The fifth step is to decide whether the computed displacements within the dam can be accommodated by the initial design of the dam. Such remedial measures as changing the dimensions of the filter zones and other features of the dam are assessed to assure the long-term stability of the dam after the internal displacements have occurred. Ambraseys indicated that steps 3, 4, and 5 are first performed during feasibility studies and, as more accurate field data and strength tests become available, the design should be modified to account for more realistic material properties. The initial analyses discussed above give conservative results for relative displacements in the dam.

31. When material strengths and shear-wave propagation velocities are known for the foundation and embankment, Ambraseys recommended that a dynamic analysis be performed using damping, about 5 percent, and radiation. A lumped mass model or a wave propagation model with radiation is suitable for this purpose. Input motion should be from an actual earthquake record scaled to the design particle velocity for the project. The purpose of this analysis is to find the peak acceleration that would be generated in the dam or its foundation. The lumped masses representing the embankment and foundation are defined from the period of a triangular shear beam.

32. In response to other questions, Ambraseys noted that the ring shear test would be useful for measuring the large strain behavior of fill material. He said that there needs to be balanced progress between the accuracy of analysis methods and the reliability of material behavior parameters during earthquakes in order to better assess displacements in dams during earthquakes. On other questions, Ambraseys said that he generally prefers steel to reinforced concrete for structures because there is less variation between the design structure and the actual structure where steel is used.

33. Ambraseys also mentioned that during laboratory tests he uses the peak value of the pore pressure parameter A for conservative estimates of embankment displacements during earthquakes. The key element in determining the embankment displacements is the cumulative time during which the safety factor of a sliding mass is less than unity. Some questioners asked Ambraseys about means for identifying liquefiable zones in foundations and he replied that the Japanese have considerable data relating standard penetration tests to liquefied zones during earthquakes. Ambraseys suggested that, for long low dams, it would be useful to check the effects of an earthquake parallel and perpendicular to the axis of the dam. He suggested that attention be devoted to analyzing buildings that are still standing after earthquakes, not only those that have failed. Ambraseys also said that one earthquake record at some location may not have the same spectral content as the next earthquake recorded at the same spot and that the spectral characteristics of the motion depend on the propagation direction of the fracture along the fault.

PART II: SUMMARY OF DISCUSSION
27 APRIL - 4 MAY 1973

Geological and Geophysical Investigations
27 April 1973

34. The discussions on geological and geophysical aspects of earthquake engineering generally followed the items listed in Appendix B. These items had been submitted to Prof. Ambraseys several weeks prior to his visit to the Waterways Experiment Station.

Surface faulting

35. Several of the items concerning geological and geophysical investigations had overlapping areas of interest and it was not possible for Ambraseys to address a single item without considering the ramification of the other items. In response to item A-1 (Appendix B) as to whether large earthquakes can occur without some surface evidence of faulting, Ambraseys said that even a small earthquake will show evidence of surface strain if sensitive instruments are used to detect the strain, but surface breaks need not develop. If it is accepted that earthquakes are generated from fault movement, then there must be some finite strain at the ground surface. The presence of sand boils in the New Madrid area does not necessarily indicate that these boils were caused by faulting in the alluvium during this earthquake.

Earthquake attenuation

36. With respect to another question, item A-8 (Appendix B), Ambraseys said that areas of the world other than the central United States exhibit a rather low attenuation of earthquake motion with distance from the epicenter. Areas in Australia and in the Himalayan Mountains (Central Daccan Plateau) also exhibit low attenuation characteristics. Ambraseys feels that there is a significant difference in the energy flux generated by faults in different areas of the world. He has noted that small stress drops occur on old faults that move often--such as those in California. Faults with long return periods, perhaps 10,000 years, usually have high stress drops. It is the high stress

drop fault that produces the greatest flux. Thus, in summary, Ambraseys said that there will always be some surface indicators of fault movements. Vertical movements, however, would be more difficult to find than horizontal movements from studies of the lineaments in the area.

37. With respect to question A-8 (Appendix B), Ambraseys said that the energy loss from earthquakes is a matter of the jointing that exists in the geological structure. For example, along a mountain chain, the propagation velocity is usually 15 percent greater than transverse to a mountain chain. This indicates that faulting, which produced the mountains, results in anisotropic earthquake motion transmission in these structures.

Significance of airphoto linears

38. Dr. R. T. Saucier (WES), who has been studying the New Madrid area, noted that sand boils can be found in distinct linear patterns. Ambraseys said this may indicate the position of either fracture or the location of old, unconsolidated, drainage channels. Strain rosettes, located in the ground, might be used to indicate the creep patterns and thus the possible location of deep faults. Ambraseys restated the concept that only competent materials will produce an earthquake; incompetent materials, such as alluvium and soft sedimentary rocks, cannot produce earthquakes but may be involved in long-term creep that results in an accumulation of strain at the ground surface. Ambraseys suggested that Dr. Lyon Sykes, Columbia University, be consulted for planning a plate tectonic study in the New Madrid area.

39. Ambraseys believes that airphoto linears may be significant as strain indicators and are worth the effort necessary for the interpretation. He said that, in the central United States, there has been little serious interpretive work. Almost everything that has been done recently has been simply a restatement of known evidence or a reorganization of past work.

Assessing fault activity

40. In response to item A-3 (Appendix B), Ambraseys mentioned that field measurements are now being used for studying a single fault and the changes on that fault with time. Gathering such data and

reporting it is very expensive and time consuming. He mentioned work by Brune, King, and McKenzie in studying faults in the LaJolla, California, area. Microearthquake records have some use, according to Ambraseys, but it takes exceptionally good instruments and skilled people to extract useful data from microearthquake investigations. He suggested that a simple earthquake counter might be used to find the area in which most of the activity is concentrated. One definition of an active fault is one that is less than 35,000 years old (U. S. Geological Survey definition). Ambraseys asked whether a fault that is older than this does not have the potential for generating an earthquake and indicated that there is no reason to believe that older faults cannot produce earthquakes.

Fault detection

41. With respect to field investigations to identify all faults in the area of a project, see item A-4 (Appendix B), Ambraseys agrees with Dr. Slemmons in that careful investigation will find faults that are only a few kilometers apart. That is, there are so many faults near the project that designers really do not know what to do about them. However, Ambraseys believes that competent geologists will find all major faults. One practical approach is to assume that the epicenter will be located at the project site and that the focus of the earthquake will be immediately under the project so that its depth will depend on the dip of the formation in the area. Also, it may not be necessary to find all the faults in any area because small faults generally lead to larger ones which may have sufficient length to pose an earthquake problem. For these situations one can consider, from the length of the faults and their proximity to the project, the return period and magnitude relationships for the faults of interest and determine whether an earthquake on a large fault or a smaller, local fault would be critical to the facility.

Seismic motion and fault type

42. With respect to the expected seismic motions from various types of faults, item A-5 (Appendix B), Ambraseys indicated that this question is purely academic. There is not enough evidence or data on

the motion generated by different types of faults to draw any empirical conclusions as to whether thrust faulting or strike-slip faulting is critical. Ambraseys indicated that, potentially, stress drops on thrust faults are usually larger than on strike-slip faults; thus, higher potential velocities would be expected from a thrust fault. He feels that the Pacoima Dam record could be produced by a strike-slip fault as well as by a thrust fault. The major parameters for earth motion effects arise in the frequency content and duration of the ground motion which is a function of the direction of propagation of the slip along the fault relative to the observer. These frequency and duration characteristics are due to the Doppler effect. Ambraseys indicated that there were similarities between the Pacoima record and records obtained during the Niigata earthquake in Japan.

Geophysical studies

43. With respect to geophysical studies, item A-6 (Appendix B), Ambraseys said that such studies can be used to assess radiation effects at the project site. When an earth dam is constructed at a site and an earthquake occurs, a portion of the seismic energy transmitted to the base of the dam can be reflected or radiated back into the foundation. This effect is characterized by the impedance ratio of the dam and foundation materials, where impedance is defined as the shear-wave velocity times the mass density of the material in each stratum. Studies of refraction methods and shear-wave velocities have been reported by Ben Howell at Penn State University and by Prof. Martin Duke and his students at UCLA. Mr. David Leeds, Dames and Moore, Inc., is also a good source for geophysical investigation techniques. With respect to the use of such methods, Ambraseys indicated that the motion attenuation properties of the ground are also important geophysical parameters and should be measured. To study attenuation, he likes to sum the energy content of the recorded motion at two or more distances from the source of the motion and then assess the energy loss as a function of the distance from the source. This method takes account of the total loss in energy, and not simply the degradation of peak motion values as is often done in practice. He indicated that a

successful means of handling ground motion records is to enlarge the record photographically on a grid paper and then to digitize the coordinates of this enlarged record.

Seismological Aspects
27 and 28 April and 2 May 1973

Baseline correction for accelerograph records

44. Ambraseys described in detail one procedure being used at Imperial College for making baseline corrections of strong-motion accelerograph records. Because of its simplicity, the procedure may be applicable to WES activities. The various steps are as follows:

- a. The record is enlarged about 10 to 15 times onto a gridded wall so that the entire record can be studied and visualized. (The essential element in this technique is a high-quality projector that can enlarge the record greatly without distortion.)
- b. The enlarged raw record is digitized manually by two people, independently. This is a rapid process because of the large size of the record and can be done in about half an hour.
- c. The uncorrected record is integrated to obtain velocity and displacement.
- d. The velocity record is visually inspected for "kinks."
- e. A baseline correction is fitted visually to the velocity record.
- f. The record is corrected.

Ambraseys said that filtering techniques are also good for eliminating jumps in the record.

Peak ground velocities and faulting

45. Ambraseys discussed the seismological aspects of earthquake engineering with particular emphasis on the New Madrid area because of WES's current interest in it. After briefly discussing reports and papers concerning the New Madrid area, Ambraseys outlined some of the activities at Imperial College and his views on approaching

seismological problems. Ambraseys' group has studied 135 earthquake records and computed the energy radiation for these records. Peak acceleration values for these records ranged from 0.07 to 0.1 g; there was no separation of motion recorded on rock or alluvium. From this earthquake study, Ambraseys' group plotted the values of peak velocity versus distance from the focus of the earthquake.* For magnitudes from 5 to 7, they found that the following empirical equation could be used for determining the maximum ground velocity.

$$\log_{10} v = 4.02 + 0.72M - 0.5 \log_{10} (11.5M - 53.0) - \log R$$

where

v = bedrock particle velocity (cm/sec), $v_{max} < 140$ cm/sec

M = magnitude of surface waves (Richter magnitude)

R = focal distance (cm), 10 to 50 km

46. Fig. 1 showed a plot of particle velocity versus distance for various earthquake magnitudes. Ambraseys has found these relationships to be valuable for assessing peak particle velocities, which are then used as the basis for establishing acceleration values expected at specific distances from the focus. In order to establish regionalization concepts for specific geographic areas, Ambraseys suggested that the work of Prof. Allin Cornell be used. Essentially, this method establishes an earthquake return period versus peak particle velocity expected for a given area. The concept combines the geometry of the fault system and the location of the project site with respect to the fault system. Cornell's work is published in the Bulletin of the Seismological Society of America, Vol 58, p 1583, published in 1968. Ambraseys is optimistic about Cornell's approach and feels that it is a practical approach for beginning a seismic risk study. He also stated that a paper by Prof. Sarma in Tectonophysics, Vol 11, 1971, describes the effect of a moving fracture front on the time history of motion expected at a given site. He indicated that the Australians have accepted the Cornell method for regionalizing sections of their country.

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* See Fourth European Symposium on Earthquake Engineering, 1972.

47. For attenuation of earthquake waves with distance from the source and with depth of focus an interesting method has been proposed by Kovislegethy. This work was published in 1960 by the Seismological Institute of Vienna. Charts have been prepared that relate the change in earthquake intensity with distance from the epicenter as a function of the depth of the focus. Thus, if earthquake intensity records were prepared for several distances from an epicenter, the depth of the epicenter could be calculated using this method.

48. Ambraseys distributed copies of a paper published in the Proceedings of the Third European Conference on Earthquake Engineering entitled "Factors Controlling the Earthquake Response of Foundation Materials" which he had written; this paper is included as Appendix C.

49. Appendix A, entitled "Behavior of Foundation Materials During Strong Earthquakes," describes an empirical study of the relative displacement of slopes subjected to earthquake accelerations; this paper was also written by Ambraseys.

50. Empirical relationships between the magnitude of an earthquake and the peak acceleration recorded during that earthquake have been shown to be less reliable as more records of earthquakes are obtained. That is, more data indicate that such relationships are less valid. Ambraseys feels that, as far as the central United States is concerned, there are no conclusive data available from which to predict the occurrence and magnitude of future earthquakes.

Fault type and risk

51. Ambraseys noted that his studies of a very few thrust faults indicate that the energy density propagated in these faults is somewhat larger than for normal and strike-slip faults. If a fault has moved once, there is no way of determining whether that fault or another associated fault will move during the next earthquake. Ambraseys warned that statistical studies should be relegated to an appendix in risk study reports as they are interesting considerations but should not be considered too important for designing specific facilities. In response to a question as to how a time history would be prescribed at the Los Angeles Dam from an earthquake occurring on the San Andreas fault,

Ambraseys again suggested the use of Cornell's geometric statistics in combination with his empirical equation relating peak particle velocities, earthquake magnitude, and distance from the focus.

Ground motions

52. In a discussion of the ground motion that can be propagated during earthquakes, Ambraseys said that there was cracking of the ground surface during the Parkfield earthquake and that he believes that the Managua earthquake produced ground surface accelerations in some areas which exceeded 1 g. If causative faults for an earthquake at a facility cannot be found, it may be assumed that the focal depth of an earthquake in California would be 5 km. If several earthquake records for magnitude 5-1/2 earthquakes or less are available for a major fault, one may use these data to extrapolate to a design earthquake up to a magnitude of 7. Ambraseys believes that the important ground motion parameter is particle velocity and that it is not necessary that this particle velocity be coincident with some recommended peak acceleration value. Peak accelerations can be obtained from the scaled particle velocity history. He also believes that acceleration records from actual earthquakes should be used for scaling to the design particle velocity.

53. Ambraseys does not think it is feasible to compute ground motions on the basis of either strain or stress relief measurements near actual faults; this is in response to item B-4 (Appendix B).

54. In response to item B-5, Ambraseys does not believe that time histories should be computed that correspond to specified values of peak acceleration and peak velocity because these are both time and site dependent; also, the frequency content is biased depending upon the fracture propagation along the fault. He also noted that strong-motion instruments do not capture frequencies greater than about 25 cps. Some of the newer instruments are capable of recording frequencies up to 75 cps, but they are not yet available for field installation.

Transfer functions

55. With respect to item B-8a, b, and c (Appendix B),

Ambraseys said that he has not worked with transfer functions very much, as his main interest has been in the nonlinear response of dams and foundation. Transfer functions would result in distortions in the frequency content and high values of resulting motion predictions. Ambraseys believes that the transfer function approach is useful if linear behavior of the materials can be expected.

Aftershock measurement

56. In response to item B-8d (Appendix B) Ambraseys believes that it is very important to collect aftershock data from earthquake. One instrument should be located at each end of the fault zone and one instrument at the center. With this arrangement, an event counter can be used to match the records from each instrument. Ambraseys uses an SMA-1 unit from Kinemetrics for this purpose because they are light in weight and self-contained. These units have been adequate for aftershock measurements. Ambraseys said that studies of topographic effects on earthquake motions are pretty complicated and that it takes very good instruments to make accurate field measurements of topographic effects. He believes that it is difficult to find capable people who understand these topographic problems and that it is an area that is too complex for Corps of Engineers participation.

Laboratory Testing
2 May 1973

Liquefaction

57. Ambraseys' opinions on liquefaction have not changed much since his last visit; if a material dilates during shear, it does not liquefy. Ambraseys said that he can understand how particle reorientation during cyclic loading can produce positive pore pressures. Cyclic load testing may segregate the sample into dense and loose zones so that the strength of the sample would depend on the loosest zone.

58. Ambraseys does not think that undrained cyclic loading triaxial tests represent the field environment during an earthquake. In cyclic tests, energy is constantly pumped into the sample, but the

sample does not radiate energy. He suggested that the energy density put into a cyclically loaded sample be compared with the energy density absorbed in an embankment using the finite element analysis procedures. (This is a topic WES might investigate.) Finn's work on shake table testing of soils is considered appropriate by Ambraseys, and he believes that this is one way to get a comparison of material behavior using various test methods.

Damping

59. One of the difficult parameters to establish for soil materials is the damping capacity. In free vibration, damping may be from 3 to 7 percent at a strain level of 10^{-4}. Ambraseys suggested that laboratory specimens be subjected to earthquake-like stresses at the base and that the top of the specimen should be free to exhibit its responses to the base input. Different lengths of specimens might be used to establish different response characteristics. As yet, there are no good methods for determining damping for laboratory specimens subjected to earthquake stress levels. There are other means of testing soil in a laboratory. Ambraseys mentioned work by Dr. Shaal in which a cyclic shear device was used and pore pressure measurements were made. Ambraseys will send WES a copy of Dr. Shaal's thesis. Also, Dr. Wroth is doing research on the energy absorbed by soil as a failure surface is generated by a slip surface through soil.

Soil properties

60. Ambraseys currently believes that the best laboratory test for earthquake behavior is a monotonic test with pore pressure measurements. Back pressure saturation should be used and the A parameter at failure, A_f , should be computed. If A_f is negative, a value of zero should be used, but the actual value of A_f should be used if A_f is positive. He feels that monotonic tests are realistic because the displacement along a failure surface on an earth slope is in one direction; thus, the laboratory test should also be in one direction. Also, actual field failure is not a repetitive displacement once slip begins. Ambraseys does not feel that cyclic tests can be used to determine strength parameters because the test is uninterpretable.

Ambraseys suggested that soil samples be cyclically loaded to near liquefaction and then a monotonic load be applied to fail the specimen in order to more realistically represent the field displacement conditions. Additionally, if the permeability of a thin loose zone is approximately 10^{-3} to 10^{-4} cm/sec, this material can dissipate pore pressures during the earthquake if it is not confined by impervious materials. Ambraseys encouraged the study of alternative methods of testing samples in the laboratory so that comparisons of the energy absorption and stress history can be made and interpreted in terms of material behavior during earthquakes. Ambraseys asked if there were any objections to cyclic testing in the United States. He indicated that an ASCE paper is soon to be published on the failure of a triaxial specimen using the finite element method. An example of the cyclic loading data obtained at WES was examined by Ambraseys. These data were from a test on material from the Newburgh Lock and Dam site. He suggested that the spikes on the pore pressure versus time records might be analyzed to find the strain that would occur during free fall of the sand grains when the cell pressure is equal to the pore pressure and the piston load was negligible.

61. Ambraseys suggested that WES might try to cycle the pore pressure instead of the deviator load during cyclic tests to eliminate work effects on the asperities of the sand grains and to study stress strain behavior under these conditions. Ambraseys also suggested that WES might cycle the load on various specimen geometries by varying the length/diameter ratio of the specimens. Such tests would show the effect of shear zone volume to total specimen volume on the development of pore pressures.

Seismic Analyses at Imperial College
3 May 1973

Earth dams

62. Ambraseys in reviewing the progress that he and his group at Imperial College had made in the area of seismic analysis of earth dams during earthquakes said that some of their efforts had been directed

toward sorting out the Morgenstern-Price method of analyzing the stability of slopes for static conditions. These topics included noncircular arcs, the generalized method of slices, and the addition of inertia forces which cause the sliding mass of the slope to develop a factor of safety of unity. Prof. Sarma (Imperial College) has looked at the effect of slice geometry: radial and nonvertical slice faces. He found that the orientation of the slice side makes little difference on critical acceleration values required for a safety factor of one. For these slices, each one must have a factor of safety greater than or equal to one for at least 95 percent of the sliding mass in order to find an accurate measure of the critical acceleration, k_c, for the entire mass. Generally, it is not possible to use the static failure surface as the critical dynamic surface for this analysis method. Also, in most cases, a circular failure surface is not appropriate for the evaluation of k_c; a circle overestimates k_c values and, hence, gives unsafe k_c values. For an earth dam, it is necessary to evaluate k_c for failure surfaces having a depth of sliding located at about 10 equally spaced intervals between the crest and the base of the dam; failure surfaces extending into the foundation can also be considered. When this has been done, a plot of the critical acceleration coefficient versus depth below the crest of the dam can be constructed; this should be done for both upstream and downstream sides of the dam. Prof. Sarma has prepared a paper for <u>Geotechnique</u> on the general subject of computing critical accelerations, k_c, using the method of slices. Sarma has developed a procedure for computing k_c manually.

63. After the seismic resistance pattern has been determined for the dam and foundation (i.e., the value of k_c for various depths of the sliding surface below the crest of the dam), it is necessary to compute the expected accelerations imposed on the dam by an earthquake. The small-strain shear modulus of the foundation material and the dam or test fill is determined by cross-hole geophysical investigations. The natural period of the dam and foundation is calculated from closed form solutions published by Ambraseys in a paper entitled "The Seismic

Stability of Earth Dams," Second World Conference on Earthquake Engineering, Tokyo, 1960. Next, the dam and foundation are represented by a lumped-mass, shear-beam model. The intermass stiffness is defined by the shear modulus of the dam or foundation at appropriate elevations. The lumped masses of the dam and foundation are determined from the failure surface locations assumed in calculating k_c values and the density of the embankment and foundation material. The number of lumped masses representing the foundation is determined by trial and error until the natural period of the lumped-mass model agrees with the natural period of the system calculated from the closed form solution.

64. An elastic-plastic, load-deformation relationship is used in the lumped-mass model to describe the stress-strain behavior of the material in the dam and the foundation. The elastic modulus is defined by the shear modulus of these materials, and the residual strength is defined by the critical acceleration required to initiate sliding on the failure surface.

65. With the mass and stiffness matrix prescribed for the model, the base of the model is excited by the horizontal component of the design earthquake acceleration-time history. The residual displacements that occur in the model during the earthquake are computed, and the prototype dam deformations due to the earthquake are inferred. Usually, a time-step of one quarter of the period of the input motion or of the smallest natural period of the dam is used in the analysis to assure a converging solution. Ambraseys has used this analytic model to study several earthquakes and their effects on the displacement of slopes. The number of modes used should be one fourth the number of masses. Plots of the residual displacement of the slope versus the ratio of the critical acceleration to the peak earthquake acceleration have been prepared from the calculated data. On such a plot, the analyzed data for earthquakes and nuclear explosions fall between the Pacoima and the Koyna earthquake displacement values. In response to questions, Ambraseys said that the lumped-mass model does not print out elastic stresses obtained during the earthquake, but these could be extracted. He said that this method was used in his report to WES on the study of an

experiment at the Nevada Test Site and also on the Poaechs Dam in Peru. It is an approximate engineering approach that can evaluate the effects of radiation of seismic energy from the dam back into the foundation.

66. Ambraseys suggested that the finite element programs developed by Seed and his colleagues at Berkeley could be used to find the time history of normal and shear forces, E and X forces, which act on the sides of slices using noncircular failure surfaces. Investigators at Mexico City have been working on this (Resenetis and Covarrubias).

67. The Newmark (University of Illinois) rigid-plastic method has not been pursued at Imperial College, and Ambraseys has not been in touch with the University of Illinois since his work there prior to 1965. Ambraseys noted some similarities between the curves and equations he has developed and charts presented in the Newmark-Rosenbluth book on earthquake engineering. He said that his charts and equations for residual displacements give conservative (greater displacements) results when compared with the results of calculations using the lumped-mass model described above.

68. In discussing laboratory testing, Ambraseys recommended that specimens be consolidated anisotropically, then monotonically loaded in an undrained condition with pore pressure measurements. The desired A factor corresponds to failure (i.e., A_f). For a material with $c' = 0$ and slope angle β , $\sin \beta = (\tan \psi'/FS)$ and $k = (\sigma_1/\sigma_3) = (1 + \sin \beta)/(1 - \sin \beta)$. This is the maximum value of σ_1/σ_3 to be used during anisotropic consolidation.

69. Various miscellaneous topics discussed by Ambraseys included:

a. Water wave effects on earth dams as reported in a Bureau of the Interior Professional Paper on Hebgen Dam. These studies included model studies of seiches.

b. Kinemetrics SMA-1 units connecting master-slave or master-master for starting purposes.

c. A thesis on transient pore pressures in shales by Tomves, who is now with TAMS working on Tarbela Dam.

Residual Dam Displacements
3 May 1973

70. As an instructional exercise, Ambraseys was asked to illustrate the use of his equations and charts for calculating the residual displacements that might develop in Lopez Dam in the Los Angeles District. The dam was assumed to have the following properties:

Height = 50 ft

Slope = 1 on 2

Debris depth = 25 ft

Crest width = 20 ft

Dry reservoir

Shell, core, and debris: $\phi' = 40$ deg, $c' = 0$.

The shells consist of silty gravelly sand and the core consists of silty sand. Ambraseys' equation and 1-D lumped mass program charts were used to illustrate differences in computed results. The critical or maximum permissible acceleration is

$$k_c = \frac{\tan \phi' - \tan \beta}{1 + \tan \phi' \tan \beta}$$

where

ϕ' = the angle of internal friction of the material

β = the slope angle of the dam

For the Lopez Dam

$$k_c = \frac{\tan 40^\circ - \tan 26.5^\circ}{1 + \tan 40^\circ \tan 26.5^\circ} = \frac{0.84 - 0.50}{1 + (0.84)(0.5)} = \frac{0.34}{1.42} = 0.24$$

If the acceleration during the earthquake exceeds 0.24 g, permanent displacement of a portion of the slope will occur. The peak bedrock acceleration for the earthquake record proposed by Seed for this dam site is about 0.65 g, the peak particle velocity is 40 ips and the maximum displacement is 18 in.

Case I: Groundwater level
<u>10 ft below natural ground surface</u>

71. <u>Method 1: Design charts (1968) for rigid plastic material behavior.</u> Using Ambraseys' chart for pore pressure parameter $B = 0$ and for side slopes of $S = 1$ on 2 (see fig. 2), and extrapolating beyond the chart values shown, $u/KgT^2 = 1.0$ for $K = 0.65$ and ϕ' $= 40$ deg. Noting that KgT, the specified or design ground motion velocity adopted, is 40 ips and that typical values for the duration, T, of an acceleration pulse are 0.1, 0.2, and 0.3 sec, the displacement of a portion of the dam, u, is: $u = 1.0(KgT)T$.

$$
\begin{aligned}
u = (1.0)\ 40\ (0.1) &= 4 \text{ in.} \\
(1.0)\ 40\ (0.2) &= 8 \text{ in.} \\
(1.0)\ 40\ (0.3) &= 12 \text{ in.}
\end{aligned}
$$

Thus, the estimated relative displacement in Lopez Dam was estimated to be from 4 to 12 in. due to the earthquake prescribed for this site. Movements in the dam would be along the slope at shallow depth, i.e., a "slabbing" displacement of the slopes would occur.

72. <u>Method 2: Charts for elastic-plastic material behavior.</u> A second way of estimating the displacements in the dam is by the use of limiting plots computed by Ambraseys as part of a study for WES (fig. 3).* This method used various acceleration time histories from various earthquakes, as shown. The ratio of the critical acceleration coefficient to the peak acceleration prescribed for the dam site is:

$$\frac{k_c}{k_m} = \frac{0.24}{0.65} = 0.37$$

The value of c** in the ordinate values of u/c in fig. 3 is close

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* See page A6, WES Miscellaneous Paper S-71-17, Report 3, Sep 1972.

** c is a dimensionless parameter equal to $\cos(b - \phi)/\cos\phi'$ where b is slope of sliding surface derived from force polygons of sliding mass for a safety factor of 1.0; see page A9, WES Miscellaneous Paper S-71-17, Report 3, Sep 1972.

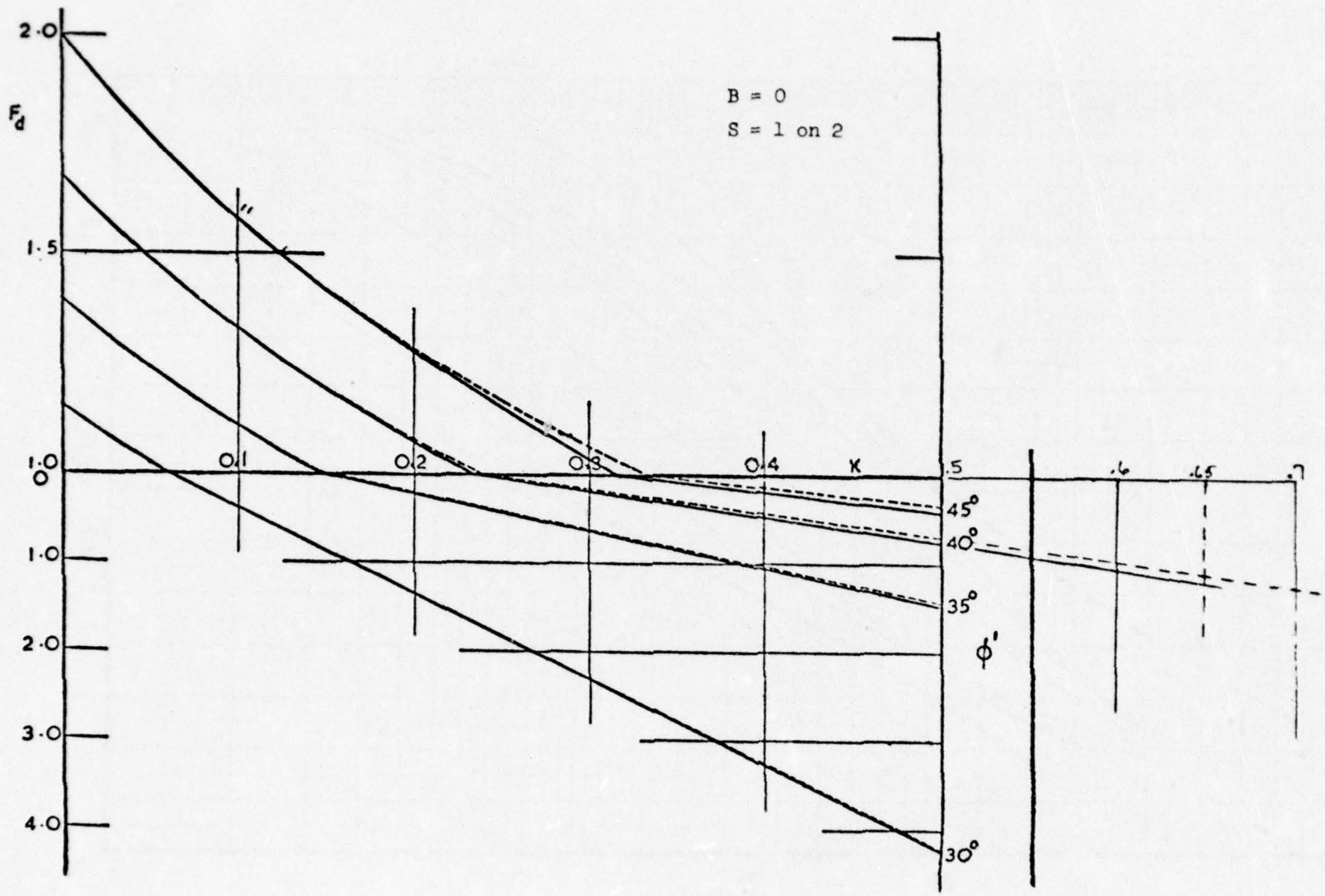

Fig. 2. Ambraseys' chart for pore pressure parameters B = 0 and side slope S = 1 on 2

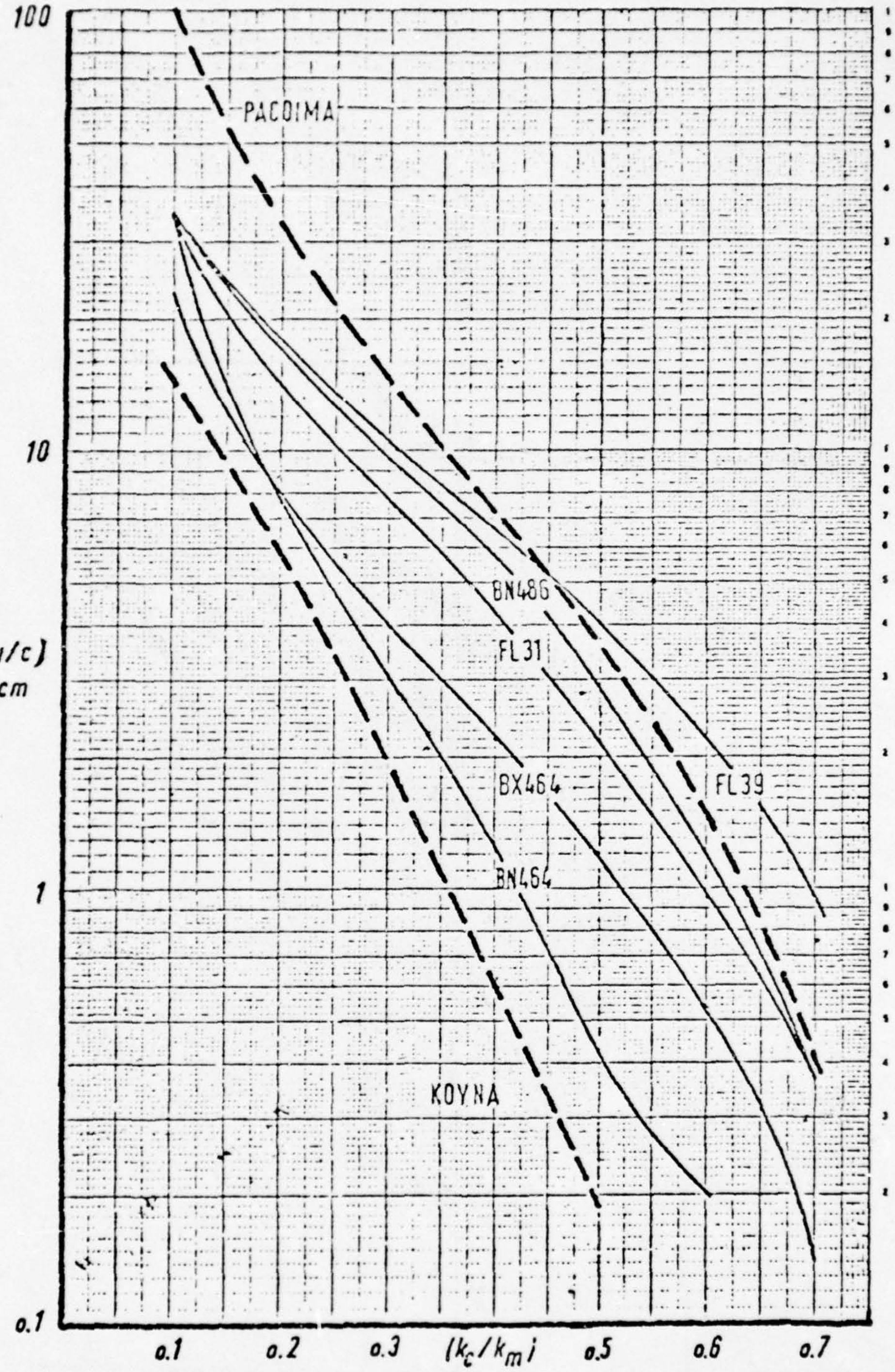

Fig. 3. Maximum displacements versus k_c/k_m

to unity. Because the prescribed earthquake was derived from San Fernando earthquake records, it is appropriate to use the Pacoima plot in fig. 3. Corresponding to a value of $k_c/k_m = 0.37$, fig. 3 gives a displacement of 8 or, since c is approximately 1.0,

$$u = (1.0)(8) \approx 8 \text{ cm}(3\text{-}1/4 \text{ in.})$$

The Pacoima record is, for practical purposes, an upper bound of the computations shown in fig. 3. The Pacoima record showed maximum accelerations and velocities at a period of 0.4 sec. Using $T = 1/2$ of 0.4 sec or 0.2 sec with the equation given under Method 1, $u = 1.0 \times 40 \times 0.2 = 8$-in. displacement for rigid-plastic analysis.

73. Method 3: Ambraseys' 1972 upper bound displacement equation. A third way of estimating the displacements in the dam is by the use of Ambraseys' empirical equation given in Appendix A and presented at the Fourth European Symposium, London, 1972.

$$\log_{10}(u) = 2.3 - 3.3 \frac{k_c}{k_m}$$

substituting, we find

$$\log(u) = 2.3 - 3.3(0.37) = 1.08$$
$$u = 12.8 \text{ cm}$$
$$= 5 \text{ in.}$$

Case II: Reservoir full

74. Method 1: Design charts (1968) for rigid-plastic material behavior. Considering that Lopez Dam had a full reservoir, Ambraseys illustrated appropriate methods of estimating displacements. Assuming pore pressure parameters of $B = 1.0$ and $A = 0.25$ for the material, estimates were made using the charts available at WES, which are for side slopes S of 1 on 2-1/2 and 1 on 4. These results were extrapolated to S = 1 on 2 ; this was necessary because the S = 1 on 2 chart was not available at WES. The charts used are shown in figs. 4

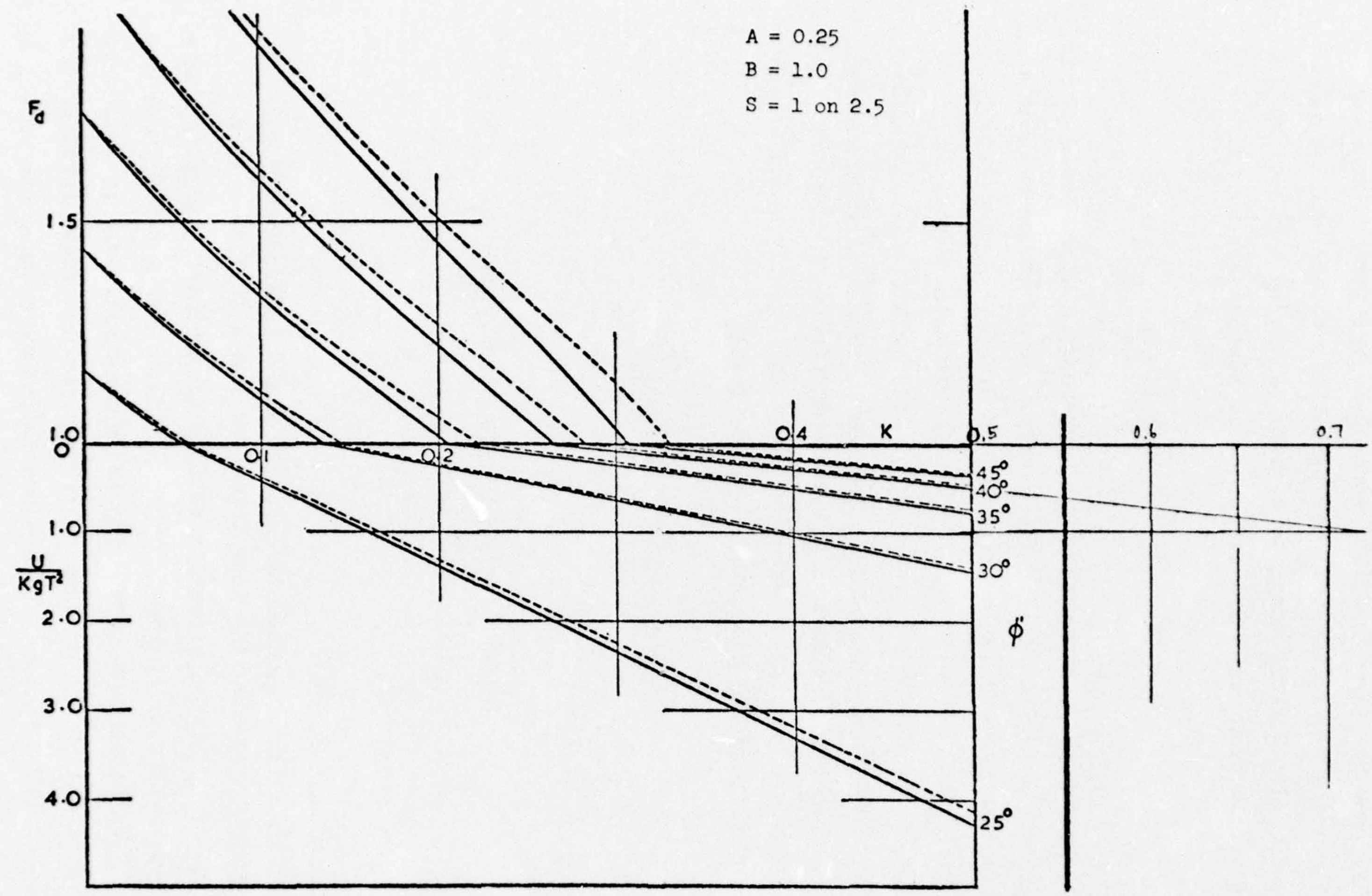

Fig. 4. Ambraseys' chart for pore pressure parameters A = 0.25 and B = 1.0 and for side slope S = 1 on 2.5

and 5. For $S = 1$ on 2.5 (see fig. 4) and following the steps illustrated above, it can be seen that permanent movement begins to develop when $k_c = 0.26$ and that for the design value of k_m of 0.65 that

$$k_c = 0.26, \frac{u}{KgT^2} = 0.8$$

Since the velocity $v = KgT$, as discussed previously, $u/KgT^2 = u/VT$ and $v = 0.8 \times 40$ (inches/second design value) $\times T$ and $u = 6.4$ in. when $T = 0.2$ sec, for a 1 on 2-1/2 slope.

75. For $S = 1/4$, (1 on 4), see fig. 5 and extrapolating, it is apparent that permanent deformations begin to develop at an acceleration of $K = k_c = 0.43$. For a design value $K = 0.65$, $u/KgT^2 = 0.2$ (extrapolated), and, as before, $u/KgT^2 = u/VT$ and $u = 1.6$ in. for a 1 on 4 slope when $T = 0.2$ sec and $v = 40$ in./sec, the design value. Linear extrapolation to a slope of $S = 1$ on 2 gives $k_c = 0.20$ as the acceleration at which permanent deformations begin to develop and $U = 8.2$ in. for $T = 0.2$ sec for the design acceleration and velocity.

76. Method 2: Charts for elastic-plastic material behavior. By the second method for a full reservoir, we have $k_c/k_m = 0.20/0.65 = 0.31$ for which fig. 3 gives, for the Pacoima curve, a displacement of 13 cm or about 5-1/4 in.

77. Method 3: Ambraseys' 1972 upper bound displacement equation. By the third method, $\log (u) - 2.3 - 3.3(0.31)$, $u = 20$ cm or 8 in. for a 1 on 2 slope.

Summary

78. All these methods as they were applied in the preceding application are for cohesionless materials with the computed movement constituting a "slabbing" failure of the outer slope at shallow depths; a summary of the calculations follows:

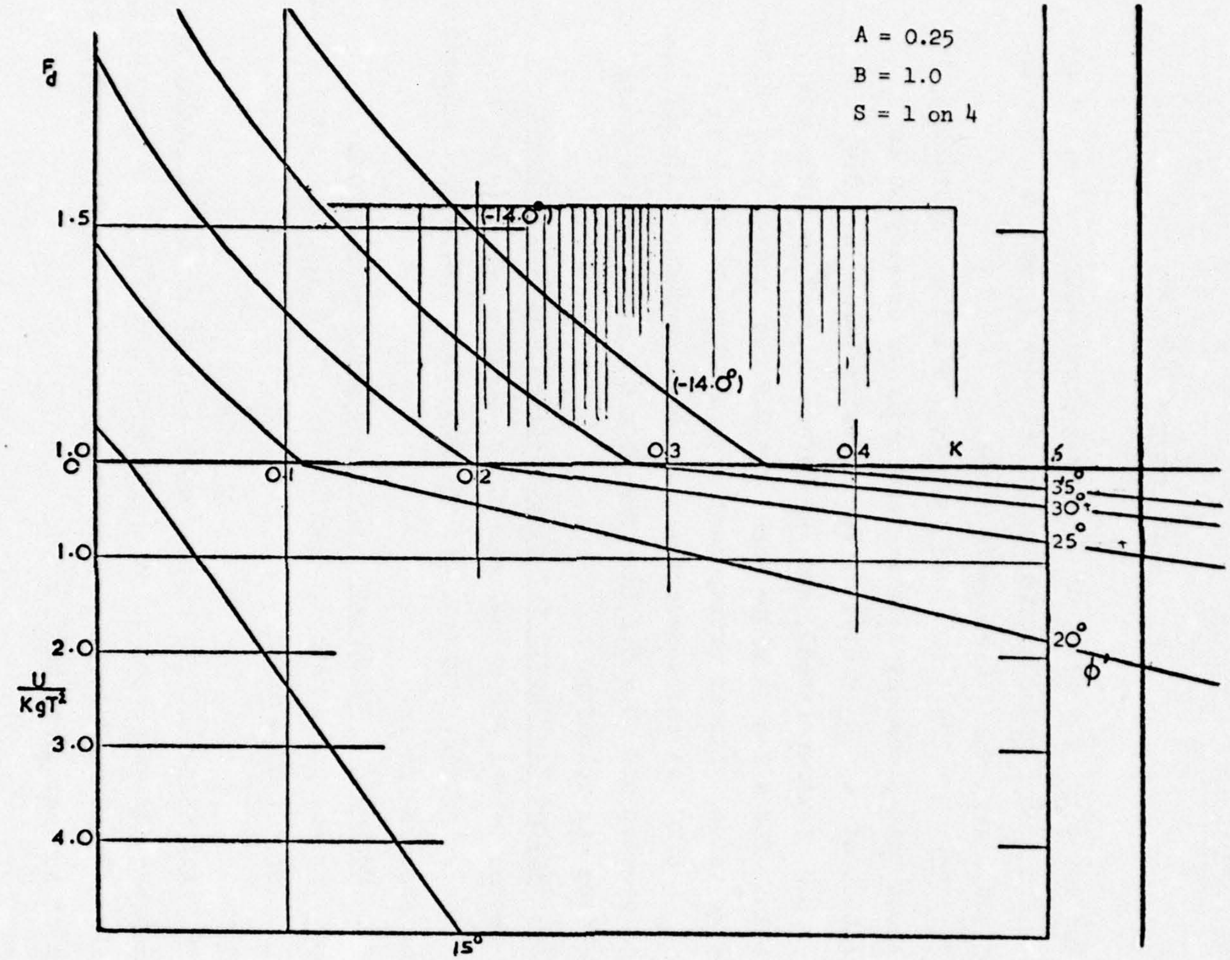

Fig. 5. Ambraseys' chart for pore pressure parameters
A = 0.25 and B = 1.0 and for side slope S = 1 on 4

Estimated Displacements (inches) of Lopez Dam
Due to the San Fernando Earthquake

Method	Case I Reservoir Empty	Case II Reservoir Full
1. Rigid-plastic material	8	8.2
2. Elastic-plastic material	3-1/4	5-1/4
3. Simplified equation for Method 2	5	8

Method 3 is actually a generalization of Method 2 and both are for elastic-plastic material. Method 1 is for a square wave loading for a rigid-plastic material.

79. In a discussion of the methods illustrated, Prof. Ambraseys said that an actual lumped-mass computer analysis would give the smallest estimate for embankment displacements, his empirical equation, Method 3, would give larger values, and his 1968 charts (Method 1) would indicate the largest displacement values.

80. For better displacement estimates, using Method 2 and fig. 3 for elastic-plastic material behavior, it is necessary to determine if the design earthquake motions correspond best to those recorded at Pacoima Dam or to those recorded at Koyna Dam. The former time history gave an upper limit and the latter resulted in minimum displacements; explosions were found to lie between these time histories, as shown in fig. 3. Since the Pacoima record resulted in an upper limit it can be used where conservative displacements are desired. However, there may often arise conditions where it is desirable to determine which curve should be used. Ambraseys suggested this be done as follows: Response spectra for acceleration and for velocity are first constructed. If the peak spectral acceleration and peak velocity have the same period, use the Pacoima line; if the difference in the periods is 0.5 sec or more, use the Koyna line in fig. 3. Interpolate between 0 and 0.5 in fig. 3 for spectral differences intermediate between Pacoima and Koyna, respectively. If this difference is more than 0.5, fig. 3 should not be used.

Cohesive embankment
and foundation materials

81. If the foundation and embankment material is only slightly cohesive, the three methods previously discussed can be used by considering the shear strength and ϕ value of an equivalent cohesionless material. An equivalent cohesionless material would have the same average shear strength as the cohesive material; hence for the equivalent cohesionless material $c'_{equiv} = 0$ and $\phi'_{equiv} > \phi'$. When cohesion strongly influences the strength of the material, it is necessary to perform stability analyses to evaluate the critical acceleration coefficients, k_c , of the dam and foundation.

82. Other hypothetical dams with more complex foundation conditions were discussed. Ambraseys reviewed the subject of limiting accelerations that can be instantaneously transmitted by shear stresses in weak foundation materials.

Miscellaneous Topics
4 May 1973

Patterns of ground cracking

83. Ambraseys described some of the patterns of ground cracking that he has observed that were due to earthquakes. He said that large arcuate patterns of ground cracking are sometimes mistaken for fault scarps; however, when a crack is traced from end to end, right lateral movement will be found at one end, left lateral at the other, and mainly throw near the center.

Influence of radiation effects

84. For the influence of radiation of seismic waves during earthquakes, Ambraseys referred to a document entitled "Non-Linear Hysteretic Response of Foundation Materials with Radiation and Material Damping," 1970-1971, which has been prepared for the National Environmental Research Council, London.* "Radiation" refers to the reflection of seismic waves from a dam or other structure back into the foundation,

* A copy of this report is available for use at WES subject to the restriction that it cannot be copied in part or in full.

thereby decreasing the amount of energy that must be absorbed by the dam or structure. Thus, radiation effects decrease the dynamic effects of an earthquake.

85. As an example of radiation effects, Ambraseys pointed out the effect of changing the impedance ratio, also called the "radiation coefficient," between the foundation and the bedrock. This ratio is defined as:

$$\text{Radiation coefficient} = r = \frac{(Sp)\ \text{layer}}{(Sp)\ \text{bedrock}}$$

where

S = shear wave velocity

p = mass density

If $r = 0$, radiation effects are present since seismic shear waves transmitted to the upper layer are reflected or radiated back into the underlying layer. If this ratio is one, which might be the case for a concrete arch or gravity dam, energy is not reflected back from the structure and is absorbed by it. If this ratio is changed by a factor of two, the acceleration levels could be changed by 30 to 40 percent.

86. Ambraseys also noted that most strong-motion instruments cannot record high frequencies so it may be necessary to convolute the record to obtain the actual ground movement. This was done for the Koyna record and is reported in Nature (1969), in an article by Ambraseys entitled "Large Earthquake Forces on a Concrete Dam." Ambraseys noted that arch dam designers, Roy Severn at Bristol University, and O. C. Zienkiewicz, have expressed concern about the validity of Chopra's Chilean paper concerning the application of the finite element method. Ambraseys noted that radiation can reduce ground response by a factor of two in general and that, for approximate evaluation of radiation effects, an earth dam can be considered as a horizontal layer. Some study has been done to find damping coefficients that would be equivalent to the radiation effect; this is included in the report "Non-Linear Hysteretic Response of Foundation Materials with Radiation and Material Damping." The lumped-mass model using the elastic-plastic material model is used on the IBM 7090 machine by Ambraseys' group.

Aftershock measurements

87. Ambraseys discussed the measurement of aftershocks and said that strong motion instruments, preferably with a 1 g rating, have been used successfully with starter settings of 5 gal. Some of the Kinemetrics SMA-1 units have vertical starters which can be triggered at 1 gal. Care must be exercised to avoid locating the instruments on slide material. The instrument should not be bolted to a base, but should be bedded in gypsum cement and leveled with a spirit level. The magnetic bearing or orientation of the instrument should be determined using a compass. A small temporary housing with a lock should be used to protect the instruments. A silica gel bag should be suspended from the top of the housing.

88. Ambraseys' measurements of aftershocks in Managua and Iran indicated peak accelerations of from 0.35 to 0.65 g and short durations of the order of 1 sec. At least three instruments should be used to measure aftershocks and they should be located about a mile apart. Ambraseys had nine battery-powered instruments, six of these are permanently located in Iran; these are situated in tight aluminum boxes with locks. Where high temperatures are a problem, he uses a foam icebox inverted and anchored around his instruments. Ambraseys said that informal arrangements are best for access to aftershock areas.

Transient pore water pressures

89. Ambraseys mentioned that pore pressure transients are being studied by B. P. Holland of Shell Development; some reports on this topic appear in Rock Mechanics Symposia (as edited by Professor Hendron); also, work is progressing at Portsmouth University (England) on propagation in three-phase porous media using finite element methods. Some porous media work is also under way in Mexico City by a student named Ajola. Pore pressure transients in a dam have been reported (1967-70) in the Journal of the Indian Society of Earthquake Engineering.

Defensive design

90. Some consideration of self-healing filters are under way by Kassif in Israel, and Wallingford at Kent University (contact G. T. Lane, Queen Anne's Lodge, S. W. London). There is an article in Geotechnique

(1968-70) on experimental techniques for flow through materials.

91. On the topic of defensive design, Ambraseys mentioned that he had prepared a plan for the operation of a dam which provided for defensive actions to be taken in the event of an earthquake. He said that much of the effort toward defensive operations for the seismic threat were thwarted by poor maintenance of the defensive facilities (stockpiles of filter material, access roads, earth moving machinery, etc.).

<u>Deformation analyses of embankments</u>

92. Returning to dam analysis topics, Ambraseys said that field modulus values from cross-hole shooting had been verified by period measurements on vibrating earth dams.

93. Regarding total-stress analyses of deformations, he said that it should be possible to calculate critical accelerations using total stresses, but that the failure surfaces would be different from those using effective stresses, which he favors.

94. In response to a question on the levels of complexity in analysis that are appropriate for earth dams, Prof. Ambraseys suggested:

- <u>a</u>. Find the critical accelerations for the dam.
- <u>b</u>. Calculate residual displacements using charts or an empirical equation for a peak particle velocity of 3 fps and acceleration spike durations of from 0.1 to 0.3 sec.
- <u>c</u>. If residual displacements are large (say 20 ft), then a more refined analysis (such as the lumped-mass dynamic model with radiation included) should be used with good field and laboratory test data.

95. If displacements of 5 ft occur without cutting the core or filter zones of the dam, catastrophic failure is not imminent. Critical acceleration calculations can be efficiently sequenced to locate the failure surface. If the static failure surface has been located and the static factor of safety is less than 2, check failure surfaces for critical accelerations just above (0.1 dam height) and just below (0.1 dam height) the static failure surface. The shear strength would not be reduced to the full residual strength of a sliding mass unless the mass moves at least a foot. Critical accelerations can be determined from

methods other than the Morgenstern and Price basic procedure, such as Spencer's method.

96. For laboratory tests on embankment materials, measurements of the relative displacements within the specimen, rather than gross specimen strain, should be attempted. The pore pressure parameter A is computed at the point where the effective stress path first touches the failure envelope, or ϕ' line. One of the problems with the Bishop and Spencer method of slices occurs on the slices near the toe where the failure surface is concave upward or horizontal; these elements cannot be analyzed for critical accelerations.

Liquefaction of sand

97. Ambraseys commented on a variety of topics involving the liquefaction phenomena. For field studies, it may be necessary to trench suspect deposits for density determinations. At large displacements in silty sand, A values may be 0.2 to 0.3. Seed's analysis of the Sheffield Dam failure is considered less than conclusive; if large strain monotonic tests on material from the San Fernando Dam had been conducted, A values of 1.2 might have been measured. Ambraseys suggested that motion measuring instruments be placed on and in material participating in an induced flow slide; he suggested an experiment on a riverbank in which the toe of the bank is progressively undercut or quickly released to induce a flow slide. (Ambraseys had suggested a similar experiment during a previous visit to WES.) He suggested other ingenious, but simple, experiments: a hammer-driven horizontal impact table to check the peak accelerations that can be transmitted by a soil layer in shear and, to calculate the critical accelerations for such a test, a rail car containing instrumented soil might be allowed to collide with a stop to generate inertial stresses in the soil. When asked who might be theoretically knowledgeable in liquefaction phenomena, Ambraseys suggested Amelio Rosenbluth in Mexico. Rosenbluth has his doctorate from Peck at the University of Illinois and is competent in structures as well. Ambraseys said that any slope stability analysis method is usable to calculate critical accelerations as long as the factor of safety is 1.0 for the sliding mass; methods, other than

Sarma's, are to be used at the risk of the person applying them. A short discussion of the shear test apparatus used by Dr. Shaal at Imperial College concluded Ambraseys open technical discussions at WES.

APPENDIX A

Session 1

1.1 BEHAVIOUR OF FOUNDATION MATERIALS DURING STRONG EARTHQUAKES

N. N. Ambraseys

Imperial College of Science, London, England

Ground deformations caused by earthquakes are either due to strong shaking or due to the low strength of the foundation materials. The field evidence on the non-elastic behaviour of soils during earthquakes is quite convincing, and an engineering solution to problems involving such behaviour must be found.

More case histories and analyses of what is likely to happen when a foundation material begins to yield are needed. Consider a natural or man-made slope, or the foundation of an engineering structure subjected to a real earthquake of maximum acceleration k_m. If k_c is the average horizontal acceleration required to bring about failure in the soil mass under undrained conditions and reduce the factor of safety to one, it can be shown that for $k_m > k_c$, the total relative displacement u in centimetres of the yielding mass will be of the order of

$$\log(u) = 2{\cdot}3 - 3{\cdot}3(k_c/k_m) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

The critical acceleration k_c is a function of the geometry of the mass as well as of the soil properties and static factor of safety of the mass profile. It is not difficult to show that $k_c = 0$ implies a static factor of safety of one, and that k_c is a measure of the reserve undrained strength of the soil mass.

Equation (1) was obtained from a numerical analysis of a variety of cases in which all available strong motion data up to 1972 were used, including near-field records from high-yield explosions; it is an upper bound solution and applies to non-sensitive soils for which k_c was calculated using residual soil strength parameters. Equation (1) is valid for $0{\cdot}1 \leqslant (k_c/k_m) \leqslant 0{\cdot}8$ and for sloping surfaces from 2:1 to zero.

It is not yet understood why strong shaking ordinarily leads to fracturing of superficial deposits and at the same time this shaking is responsible for large accelerations in the deposit. The final appearance often suggests fracturing through shear failure and it seems impossible to maintain that accelerations greater than those required to cause failure can be transmitted to the surface. For a normally consolidated deposit it can

be shown that the maximum particle acceleration because of shear near the surface cannot exceed

$$a = (c_u/p')g \quad . . . . \quad (2)$$

where c_u is the undrained shear strength at a depth where the effective overburden pressure is p'.

For normally consolidated clays, a quick estimate of the free-field maximum ground acceleration may be obtained from

$$a = [0{\cdot}11 + 0{\cdot}0037(PI)]g \quad . . . . \quad (3)$$

where PI is the plasticity index of the clay. This equation implies that low plasticity deposits are incapable of transmitting to the surface large horizontal accelerations. This is because such deposits will show small values of (k_e/k_m), which in turn implies large internal shear displacements at the expense of which strong accelerations will be deterred from reaching the surface. Thus, during strong earthquakes soft foundation materials will neither experience nor transmit severe shaking. Damage to structures on 'poor ground' is mainly because of secondary effects arising from spreading and tilting or from excessive differential settlements rather than from large inertia forces. In contrast, bedrock with high shear strength may show accelerations greater than 100% g before it fails in shear or through slabbing.

Finally, a lot of data from shallow strong earthquakes of magnitude $4{\cdot}5 < M < 7{\cdot}0$ suggests that the maximum bedrock particle velocity v, at an epicentral distance R, will be of the order of

$$\log(v) = 4{\cdot}02 + 0{\cdot}72(M) - 0{\cdot}5 \cdot \log(11{\cdot}5M - 53{\cdot}0) - \log(R) \quad . \quad (4)$$

where v is in cm/s and R is in cm.

APPENDIX B: POSSIBLE DISCUSSION TOPICS

A. Geological and Geophysical Investigations

1. Can large earthquakes, say magnitudes 6-8, occur without surface indications of displacement along known or unknown faults?

2. Should we expect to find evidence of fault movements resulting from the New Madrid 1811-1812 earthquakes?

3. Are there any recent developments in techniques for assessing fault activity?

4. Is it practicable to find all faults that might be responsible for destructive seismic effects at a dam? (Also see B-1 below.)

5. Please discuss and compare seismic motions resulting from movement along various types of faults; i.e. thrust, strike-slip, and normal.

6. What types of geophysical studies are recommended for input to seismic design investigations?

7. What are your views on the utility of geopiezomagnetic phenomena-changes in magnetic properties exhibited by magnetic rocks subject to pressure (the *Cannikin experiment*)?

8. There appear to be differences in attenuation with distance for earthquakes east and west of the Rockies. Are similar differences noticed elsewhere?

9. Can you speculate about the kind, depth, and other aspects of faulting responsible for the New Madrid earthquake?

B. Seismological Aspects

1. In areas such as California, is it sufficient to design major dams solely on the basis of careful field and associated studies in which "all" (see question A-4) causative faults are located and ground motions transferred from the fault to the site? If not, are there minimum earthquake ground motions for which all major dams should be safe? These are?

2. How can acceleration time-histories be selected for sites having no instrumental records? (Question refers to vertical and horizontal ground motion components.)

3. Please review concepts for selecting site ground motion velocities.

4. Does it appear feasible to compute ground motions to be expected at a site on the basis of either strain or stress relief measurements near faults?

5. Can acceleration time-histories be computed that correspond to specified values of peak acceleration and velocity, together with some additional information on other acceleration peaks, if necessary?

6. One approach to defining design earthquakes is to construct an envelope of response spectra for various individual earthquakes. Does this result in a compounding of safety factors and result in such a conservative definition of design earthquakes as to be inconsistent with available experience or is it a reasonable approach?

7. If a substantial earthquake has occurred (magnitude 6 or 6.5 or more), can future earthquakes be expected to occur along the same fault that would be larger in (a) magnitude, (b) peak acceleration, or (c) maximum velocity?

8. Assume that ground motions at a dam are recorded at several locations, including one on rock, during moderate earthquakes and that transfer functions are determined between the location on rock and the various other locations. Further, assume that expected rock motions for larger earthquakes, i.e. design earthquakes, have been determined:

a. Would ground motions predicted for the larger or design earthquakes on the basis of transfer functions previously determined indicate accelerations and velocities that are too high or too low?

b. Does this technique provide a simple means for making a preliminary assessment of the probable behavior of a dam during design earthquakes? (This question assumes that further detailed studies would be made if unsatisfactory behavior were predicted.)

c. For the transfer function technique to be suitable, what extrapolation might be acceptable?

d. Does measurement of aftershock ground motions at a dam where ground motions were not recorded during an event appear worthwhile as a means for developing transfer functions and for estimating a "worst case" behavior of the dam during design earthquakes?

C. Laboratory Testing

1. What soil test methods are recommended to assess soil strength and deformation behavior during earthquakes?

2. Recent work by Seed, Finn, Lee, etc., relates Roscoe-type cyclic simple shear tests to results from cyclic triaxial testing. This has resulted in the conclusion by various researchers that only cyclic

triaxial testing is required for design purposes. Do you agree? If not, why?

3. Are there any recent developments in cyclic laboratory testing of which we may be unaware?

4. Some researchers believe that the behavior of soils during earthquakes cannot be even approximately estimated from monotonic loading of undrained specimens. Do you agree?

5. How can soil strengths for use in the Ambraseys-Newmark procedure for computing permanent relative displacements be determined from cyclic triaxial compression or other cyclic tests?

6. As above, for appropriate damping values?

D. Dynamic Analyses

1. Please review the historical development and present status of dynamic analyses for computing earthquake motions of dams, discussing (a) shear slice method, (b) various 1-D programs, (c) 2-D rigid base finite element analyses with horizontal only and with combined horizontal and vertical bedrock motions, (d) traveling waves in the foundations, etc.

2. WES currently has Berkeley and MIT 1-D and 2-D dynamic programs but does not have a shear slice program. Should such a program be acquired? (Why and how?)

3. Please discuss appropriate dynamic analyses for various combinations of height and type of dam and consequence of failure. The emphasis intended is mainly on the applicability of different levels of dynamic analyses, which are envisioned as (a) generalized seismic coefficient charts derived from dynamic analyses, such as you presented in your "Geotechnique" paper and also gave us for 38 earthquakes in your 1968 visit, (b) shear slice analyses using various acceleration time-histories, (c) 2-D finite element analyses with only horizontal motions for rigid bedrock, and (d) 2-D finite element analyses with vertical and horizontal motions for a rigid bedrock. Can computer time required be discussed for each alternative?

4. Describe your present method for computing permanent relative displacements in a dam.

a. Please discuss various types of damping, appropriate damping values, etc.

b. Are there differences between your current method and that given by Newmark in his 1965 "Geotechnique" paper?

c. WES does not have a computer program for permanent relative displacements where an acceleration time-history is used as an input. You used such a program in the NTS studies you made for us. It appears that we need such a program of Lopez Dam and other uses; can it be obtained?

E. Pacoima Record

1. Is the accelerograph record considered a valid description of bedrock motions at Pacoima Dam or did it record amplified bedrock motion?

2. The maximum velocity is associated with accelerations less than 0.6 g and occurred well before the peak accelerations were recorded. Are the few long-time intervals in which the periods of accelerations were large typical, perhaps reflecting the arrival of shear waves, or are they unusual and suggestive of movements resulting from cracking of the abutment and/or shifting of the accelerograph?

F. Miscellaneous

1. Are there recent developments in "defensive design" procedures; i.e. since your 1970 visit?

2. Are there aspects of defensive design that can and should be assessed analytically?

3. Please review your concepts on liquefaction concepts and laboratory and field tests to determine liquefaction potential.

APPENDIX C

BULGARIAN ACADEMY OF SCIENCES
EARTHQUAKE ENGINEERING
PROCEEDINGS OF THE THIRD EUROPEAN SYMPOSIUM ON EARTHQUAKE ENGINEERING
Sofia, September 14—17, 1970

FACTORS CONTROLLING THE EARTHQUAKE RESPONSE OF FOUNDATION MATERIALS

N. N. Ambraseys[1]

1. INTRODUCTION

One of the main problems in seismic micro-regionalisation is the classification and mapping of soil deposits according to their properties of modifying bedrock earthquake movements. When a soil deposit is excited into oscillations, it will respond in a manner which will be dictated by the character of the bedrock motion and also by the properties and geometry of the deposit itself. Thus, there are local variations in seismic response due to differences in ground conditions. The response will depend on the resilience and capability of the deposit to dissipate energy and also to radiate part of it back into the source. The lower its capacity for energy dissipation and radiation, the greater will be its response.

Depending on the circumstances, the response may bring about an amplification or a depression of the bedrock motion. In the first instance, if the stresses induced in the deposit are well below the minimum strength of the soil, the response may be considered to be elastic. Material damping and radiation effects will then be operative, and although their combined effect in reducing amplification will be considerable, the layer response will, in general, show an amplification of the bedrock motions. In this case, however, large ground accelerations resulting from the amplification of the bedrock motion will require a strong foundation material, capable of transmitting them to its surface while the soil remains elastic.

The purpose of this paper is to illustrate the effects of radiation and plastic yielding on the response of foundation materials and at the same time determine the conditions of amplification of the soil response.

[1] Engineering Seismology Section, Imperial College of Science, London.

2. SOIL RESPONSE

Material damping, radiation effects and plastic yielding in foundation materials has a pronounced effect, not only on the layer response during

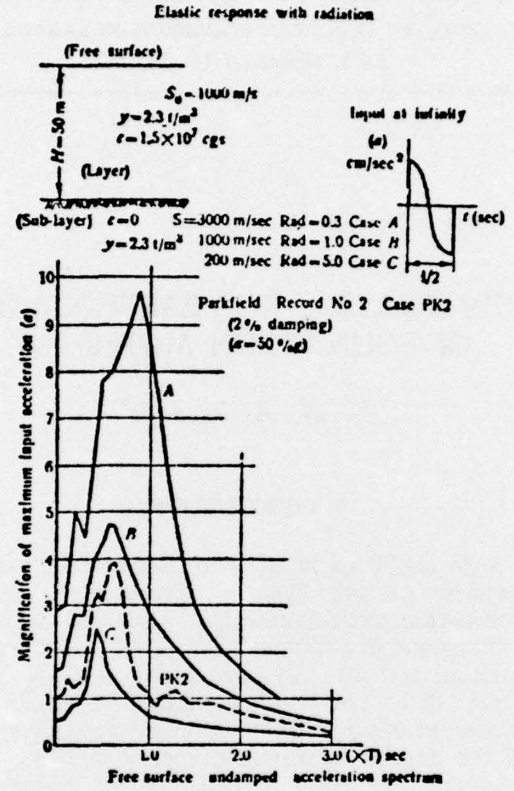

Fig. 1

earthquakes but also on the post-seismic stability of engineering structures. These three factors will, in general, tend to depress response and reduce the earthquake forces in a structure at the expense of large ground deformations which, under extreme conditions may lead to structural failures even after the earthquake has ceased.

Analytical methods of computing layer response during earthquakes so far have been based mainly on linear or bi-linear hysteritic models with no radiation.

Elastic layer response with radiation. A computer programme has been developed to evaluate the layer response with radiation, using a lumped-parameter solution [12] This programme was used to study the effect of radiation. As an illustration of these effects consider the simplified case, shown in Fig. 1: an elastic layer 50 metres thick overlies a

sublayer, the system being subjected to a half-cosine shear-wave, emerging from infinity. In this example, the numerical values are given in Fig. 1; a cosine input has been used, because it represents the type of acceleration pattern that will arise at the source from a stick-slip fault motion.

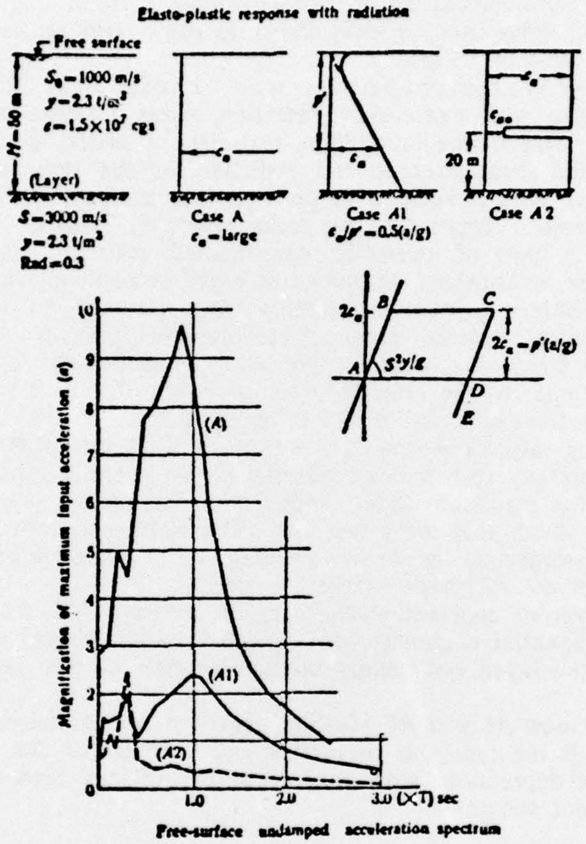

Fig. 2

The spectral response of the layer for three radiation coefficients is shown in Fig. 1. In order to facilitate the identification of higher modes, spectra were computed for zero damping. From these curves we notice that both the maximum response and the width of the frequency band over which the spectral response is magnified decrease with increasing radiation. Energy radiation can be considered to be of the nature of additional damping and its effect in depressing layer response is significant. This is particularly so, because radiation narrows the frequency band of maximum response. For instance, the reduction of peak response from nearly $10a$ for case A to about $2.5a$ for case C is less significant than the fact that for periods smaller than about 0.3 (T) and greater than 0.8 (T) the spectral response for case C is depressed to magnifications of less than a.

Energy radiation into the bedrock also can be considered to generally modify the input motions. In spite of the simplicity of the input used in this example, higher modes produce a rather complicated spectrum, which is very similar in shape to that of the Parkfield record.

The results of a study [12, 14) of the radiation effects on the soil response for real inputs show that neglecting the radiation present in seismic ground movements is very likely to cause large errors in the assessment of the layer response.

Elasto-plastic response with radiation. With increasing input amplitudes and decreasing radiation, shear stresses in a layer may bring about failure of the foundation material. In order to investigate the effect of limited shear strength and radiation on the response, a computer programme has been developed to perform this analysis. [12] Fig. 2 shows a simple example of input data for three cases: an elastic layer, Case A; an elastoplastic layer of normally consolidated material, Case $A1$; and a saturated layer of constant undrained strength containing a weak layer, Case $A2$. The acceleration input at infinity was assumed to be a half-cosine acceleration pulse. Material damping was operative only in the elastic range and the yield stress at a point for a normally consolidated deposit was taken to be proportional to the effective vertical stress. Figure 2 shows the undamped aceleration spectra for the three cases.

Comparing spectra A and $A1$, we notice that the spectral response of a layer of normally consolidated material with a strength ration (c_u/p') equal to, say, half the maximum input acceleration coefficient (a/g), will be four times smaller than that of a material of infinite strength for which the response is assumed to be elastic. Yielding of the material will have a pronounced effect on the shape of the spectrum.

For a layer of constant strength c_u, containing a thin deposit of weaker material, the spectral response will depend to some extent on the relative strengths of the layer, but mainly on the location of the layer of weaker material.

In both cases $A1$ and $A2$ yielding caused a substantial reduction of the amplification. If the radiation coefficient had been larger, the layer response would show a depression with magnifications of less than one, Cases $B1$ $B2$, $C1$, $C2$ (not shown).

3. DISCUSSION

With natural soil deposits not only radiation and material damping should be considered in the layer response, but also elastoplastic behaviour and layering. The spectra shown in Fig. 1 and 2 are only indicative of these affects on the layer response. They illustrate the complexity of the problem and show that each particular deposit is a special case for which no simple solution or rules can be adopted.

The numerical technique that has been developed offers solutions to complicated response problems, but the parameters involved are so numerous that it is impossible to present the results in a generalised form.

Maximum ground acceleration. The only general comment that can be made at this stage is that, for saturated, normally consolidated soils,

the maximum ground acceleration at the surface cannot exceed a value equal to $(c_a g/p')$. At some depth the maximum acceleration may reach, but not exceed, a value twice as large. This maximum value of the acceleration can be quite high, and it depends on the ultimate strength of the deposit-structure system.

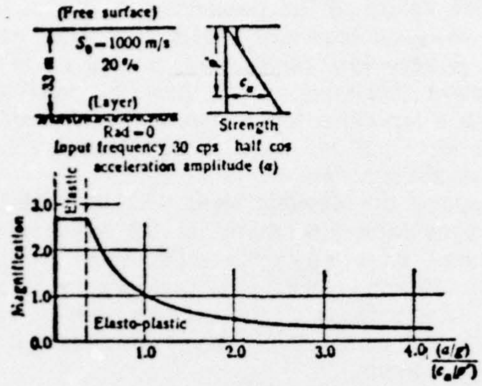

Fig. 3

The example shown in Fig. 3 illustrates this point. A layer of normally consolidated clay, 33 metres thick, with an S-wave velocity of 1 km/sec, is subjected to a half-cosine pulse of frequency 30 cps. Fig. 3 shows the magnification of the bedrock acceleration as a function of the dimensionless strength parameter of the layer $(a/g)/(c_u/p')$. We notice that for strong materials or for small accelerations, the layer will behave elastically and will amplify the bedrock acceleration a at the free surface by a factor of 2.8. For longer wave-trains the magnification can be large. However, as the strength of the deposit decreases or as the input acceleration increases, the response becomes elastoplastic and the magnification of the input acceleration decreases to values less than one for $(a/g)/(c_u/p') > 1.0$.

For normally consolidated clays the ratio of the undrained shear strength c_u to the vertical effective stress p' under which it was consolidated in the field, shows a close correlation with the plasticity index (PI). The correlation is now confirmed by sufficent field evidence to require only the determination of the plasticity index, from which the strength ratio can be estimated from the empirical formula [13, 3]:

$$(c_u/p') = 0.11 + 0.0037\ (PI). \tag{1}$$

Thus, a very soft marine deposit, which shows, say, $(PI) = 10$, should be incapable of transmitting to the surface absolute accelerations greater than 15%. Under repetitive loading such a material may be very sensitive. Thus most probably it will show smaller undrained strength ratios than those given by equation (1) and consequently it will transmit smaller accelerations. Inorganic clays of low and medium plasticity with $(PI) = 50$ will be capable of transmitting accelerations not larger than about 30% g. Deposits of high plasticity may transmit accelerations up to 50% g.

For saturated silty sands the undrained strength may be obtained from appropriate laboratory tests [3] or it may be estimated from the formulae in Appendix *I*. Cyclic loading and strain-rate effects act in an opposite sense so far as the actual value of (c_u/p') is concerned. Hence the values that are obtained from the formulae in Appendix *I* may not be excessively conservative. In these formulae the values of the parameters k and A should be obtained from appropriately designed laboratory tests. For large values of the pore pressure parameter A very low values of c_u should be expected, leading to liquefaction [4]. Ground accelerations will now be subdued to very small values, and although a structure founded on such a deposit will be subjected to small earthquake forces, it will have to withstand large static forces arising from foundation settlements.

In a layered deposit the weakest layer will control the response. In this case simple calculations cannot be made for the assessment of the response.

Field evidence of elasto-plastic layer response. Fracturing and cracking of flat ground caused by earthquakes is not an uncommon phenomenon [18, 2]. Comparatively short, open cracks, extending to some depth in flat ground, and compression ridges are features usually attributed to strong ground movements.

Yet the presence of surface cracks, irrespective of their size, indicates that the response of the deposit should have been strongly non-linear and also that it must have yielded along horizontal planes at some depth. It is

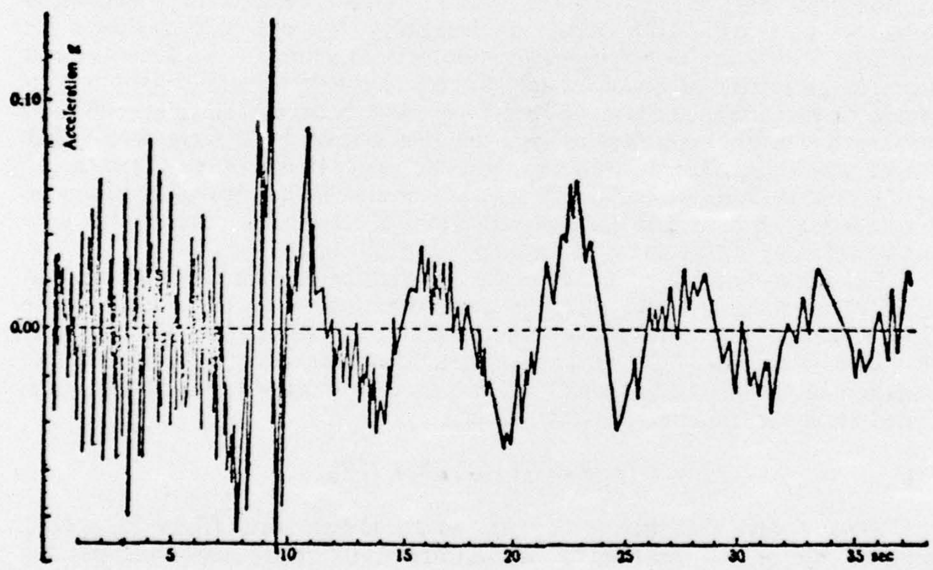

Fig. 4

difficult to see how vertical cracks and pressure ridges of any size can develop without a common failure surface joining them. It is perhaps significant that, in many instances, ground fractures, lurching and pressure ridges are associated with weak layers in a deposit or with liquefaction of these

layers [1]. Field observations also indicate that in many cases the ground motion that resulted in cracks forming in the ground was far from intense [9, 7, 5]. The damage to structures observed in these cases was due more to the discordant character of the ground motion and to large permanent foundation displacements rather than to the mere increase in intensity at the surface of the deposit, an observation made by Reid as far back as in 1910.

Fig. 4 shows the accelerations recorded in the basement of a building which lost its foundation support during the Niigata earthquake. The early part of the record shows comparatively high frequency movements that correspond to the elastic response of the foundation materials. About 8 seconds after the beginning of the recording, the foundation material was brought into its plastic range and this was followed immediately by liquefaction.

Secondary effects. So far, our field experience and the results from an extensive analysis of the layer respons suggest that poor foundation materials subjected to strong earthquakes will, in general, be responsible for heavy damage to structures founded on them. However, much of the observed damage is due to foundation failures, ground settlements, tilting and liquefaction and not due to large earthquake forces. Invariably, as the foundation material becomes less competent, damage starts from secondary foundation effects [18, 9, 5, 10, 17, 7, 16, 11].

Base-shear coefficient. Code Requirements. It is customary to attribute damage solely to earthquake forces and this in turn to poor foundations, with the obvious corollary that poor foundations imply large earthquake forces.

This attitude is reflected in many National Building Codes [6] in the form of a numerical coefficient for correction of the design base-shear, according to the foundation conditions. Thus, various Codes suggest and in some cases enforce the use of larger seismic forces for structures founded on poor foundations than for those founded on rock. The Algerian Code gives a foundation coefficient of 2.5; Chilean 2.4; Cuban 2.6; French 1.4; Greek 2.0; Italian 1.5; Iranian 1.25; Japanese 4.0; Mexican 1.5; Rumanian 2.0; Spanish 2.0; Turkish 1.6; USSR 1.5; Yugoslav 1.5.

It is difficult to see how the use of an increased base-shear coefficient will cater for damage arising from foundation failures. The use of a large foundation coefficient will result in producing a more rigid structure, which in fact will be more sensitive to differential settlements and which may be rendered useless if its foundations fail. In Niigata, for instance, little if any of the damage to buildings was the direct result of the earthquake vibrations [7, 11, 16] and little or no structural damage occurred as the result of intense ground movements that would have required an increased base-shear coefficient. The record shown in Fig. 4 shows quite clearly that the poor foundations in Niigata (for which the Japanese Code suggests an increase in base-shear coefficient by a factor of 4.0) were incapable of transmitting accelerations greater than about 15%. Most of the damage in Niigata, as well as in other earthquakes where poor foundations were involved, was due to foundation settlements aggravated by the shock; it was the failure of ground support rather than of structural elements that initiated collapse. In Alaska most of the spectacular damage was related to ground movements which caused foundation failures [17]. Structural damage in the Anchorage area resulted from secondary causes such as ground cracks and slides, them-

selves triggered by seismic vibrations; structural damage due directly to seismic vibration was subordinate [9]. In other parts of Alaska damage to structures that can be directly attributed to strong ground motion was slight and not widespread [5, 10].

4. CONCLUSIONS

For real soil deposits each case of micro-regionalisation is a special case and it should be treated as such. Radiation and elasto-plastic effects should be included in the study of the layer response.

Large ground accelerations can be transmitted to a structure only through strong deposits. For the micro-regionalisation of such deposits elastic theory and microtremor work are applicable.

Weak foundation materials will be incapable of transmitting large accelerations. In this case the regionalisation must be carried out on the basis of the minimum strength of the deposit. Existing techniques based on elastic theory and microtremor work are not applicable.

Foundation coefficients in the design of engineering structures should be used with caution. Thece coefficients will give misleading results in the case of weak foundation materials. In this case bore-hole data is invaluable.

In soft materials differential settlements and loss of foundation support is more important than ground accelerations.

Dry gravel and sands should not be used as energy absorbing foundation materials.

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Appendix 1

Undrained Strength Formulae

Isotropic Triaxial

$$c_u = \frac{c' \cos(\varphi') + p' \sin(\varphi')}{1 + (2A-1)\sin(\varphi')}.$$

Plane Strain Isotropic

$$c_u = \frac{c' \cos(\varphi') + p' \sin(\varphi')}{1 + \sqrt{3}\left(A - \frac{1}{3}\right)\sin(\varphi')}$$

Anisotropic Triaxial

$$c_u = \frac{c' \cos(\varphi') + [k + (1-k)A]\, p' \sin(\varphi')}{1 + (2A-1)\sin(\varphi')}$$

Anisotropic Plane Strain

$$c_u = \frac{c' \cos(\varphi') + \left[(1+k) + \sqrt{3}\left(A - \frac{1}{3}\right)(1-k)\right] \frac{1}{2} p' \sin(\varphi')}{1 + \sqrt{3}\left(A - \frac{1}{3}\right) \sin(\varphi')} .$$

c' — cohesion in terms of effective stresses
φ' — angle of shearing resistance in terms of effective stress
k — coefficient of earth pressure at rest
A — pore pressure parameter
c_u — undrained shear strength
p' — vertical effective stress in situ

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13. ABSTRACT

This report summarizes lectures and discussions by Prof. N. N. Ambraseys, Imperial College, London, during his visit to the U. S. Army Engineer Waterways Experiment Station (WES) from 27 April to 4 May 1973. Prof. Ambraseys' lectures on engineering seismology were attended by representatives of Corps of Engineers District and Division offices and the Office, Chief of Engineers. These lectures are summarized in Part I herein. Discussions of geological and geophysical aspects and field investigations relevant to earthquake were held with WES personnel. The sequence of the discussions followed a list of discussion items that had been previously submitted to Prof. Ambraseys. Discussions also covered the application of laboratory test data in analytical procedures, current research in earthquake engineering seismology at Imperial College, Ambraseys' method for computing dam displacements, and other topics related to earthquake engineering. The discussions with WES personnel are summarized in Part II.

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